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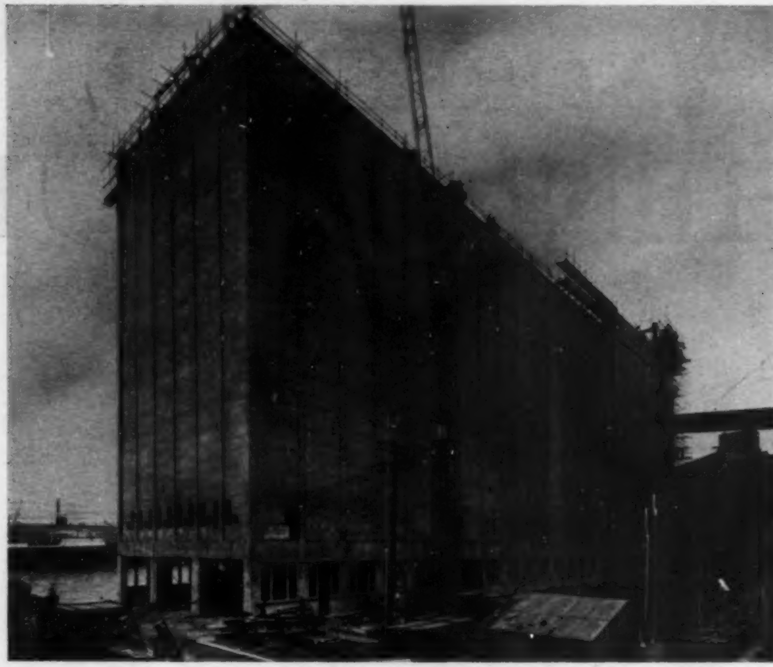

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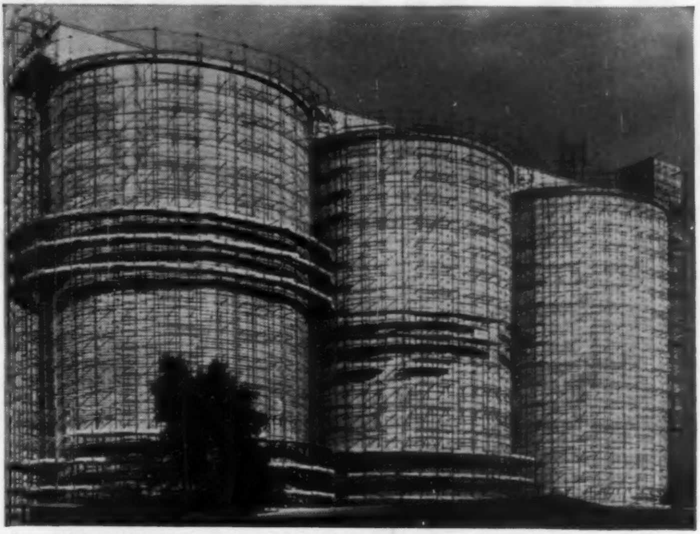
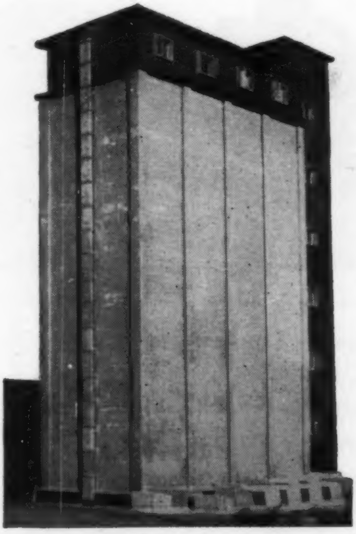


Sugar Silos at King's Lynn for British Sugar Corporation Limited

Silo construction



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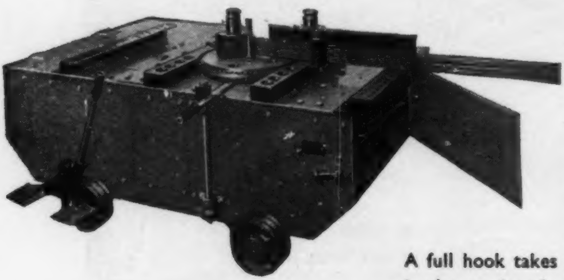

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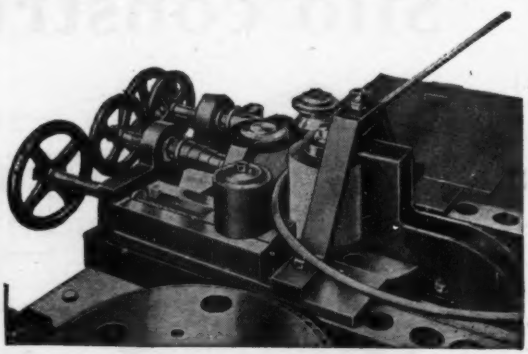

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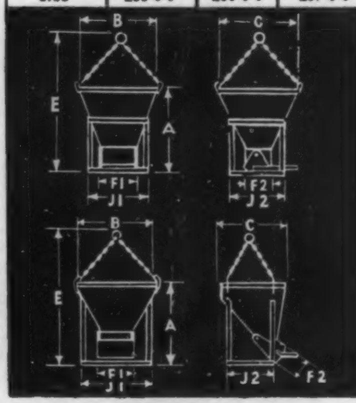

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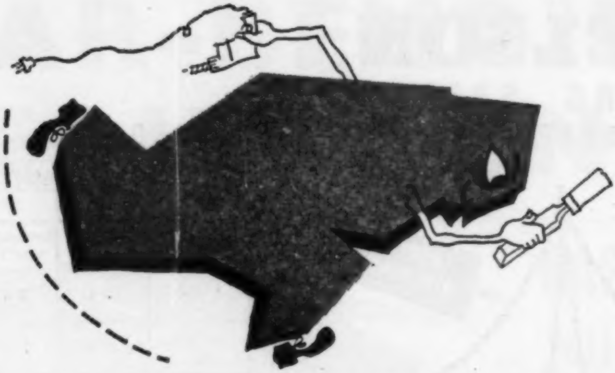

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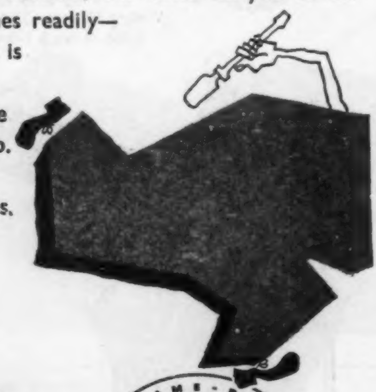

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Rawlties are for use where shuttering is erected on both sides of a wall.

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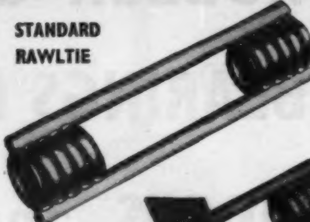

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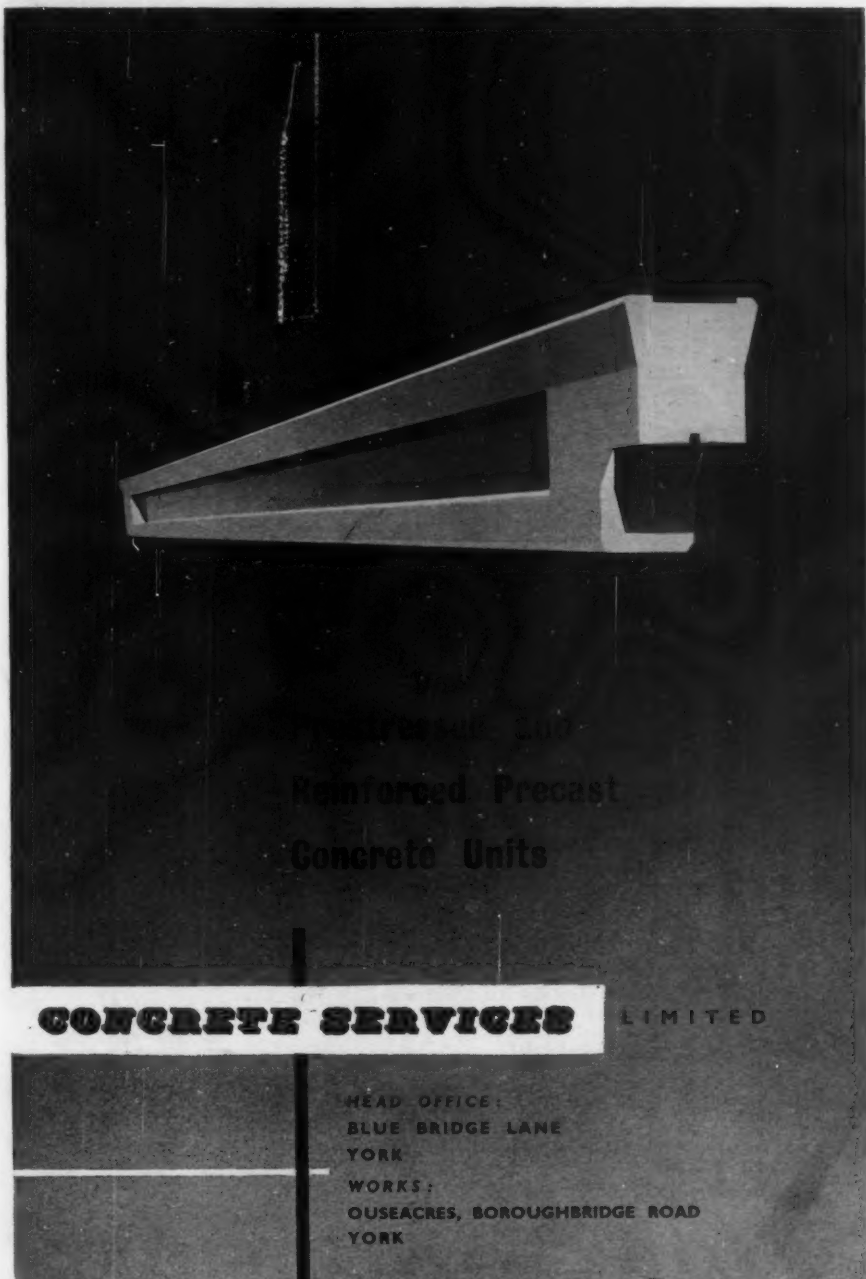

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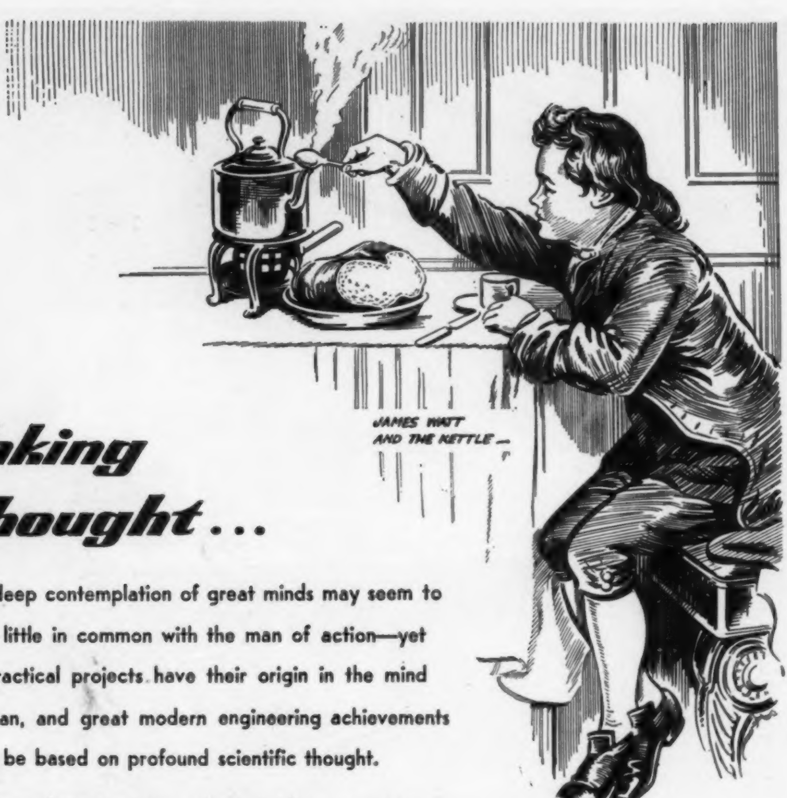

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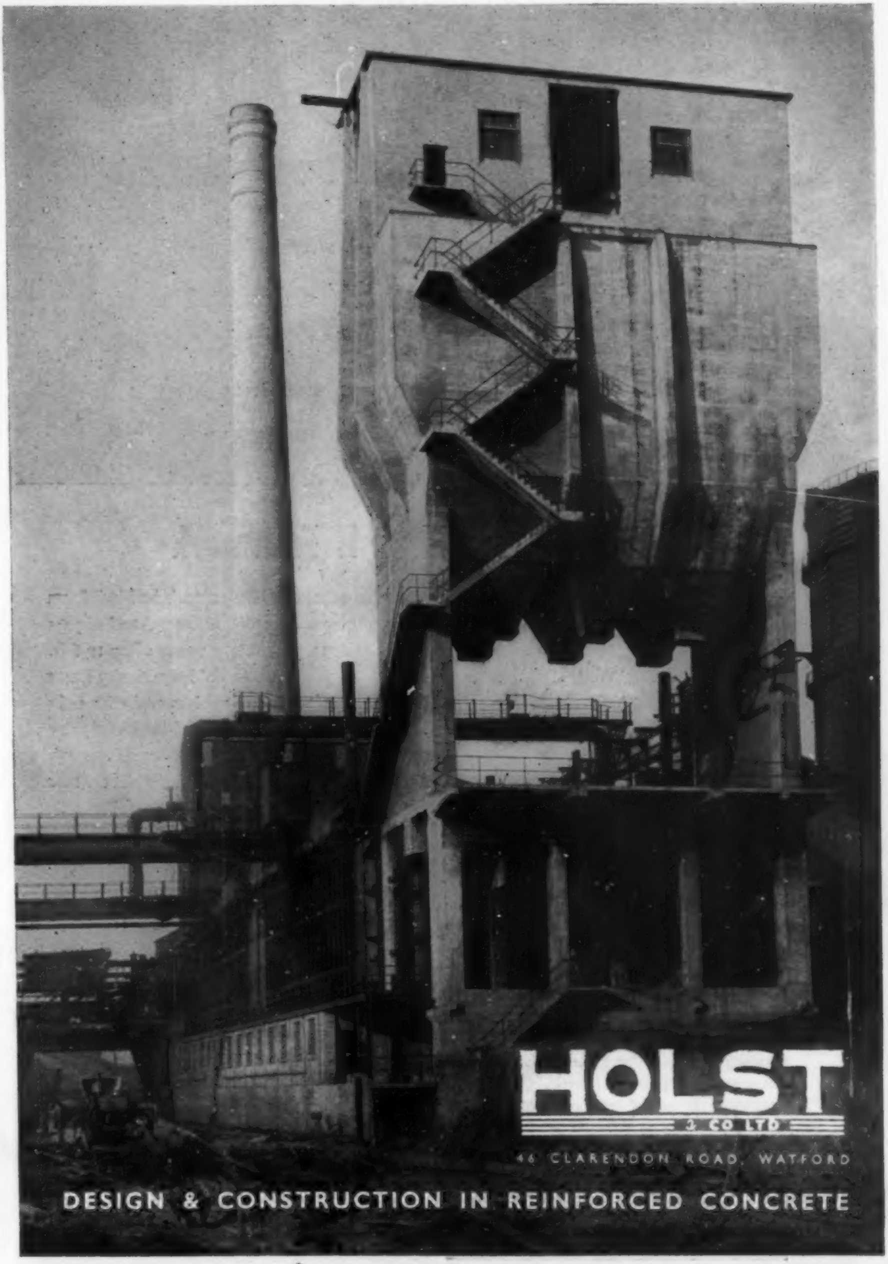

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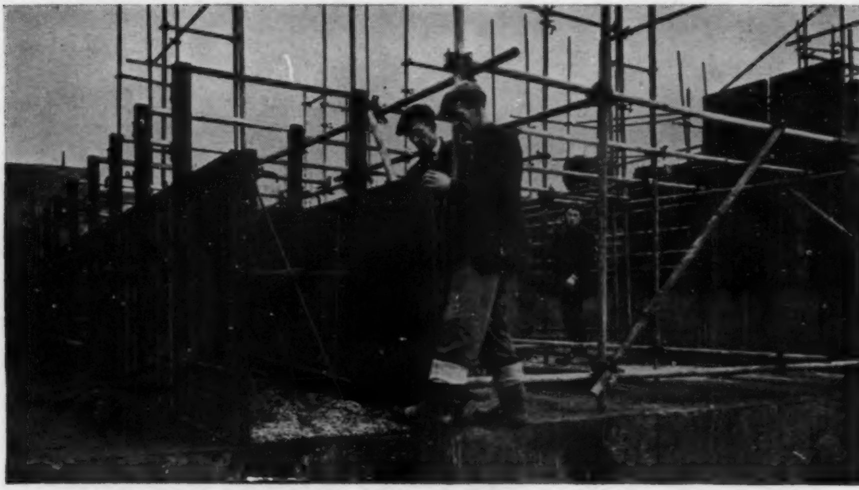

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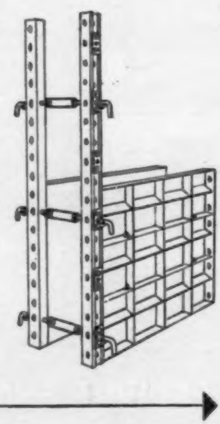

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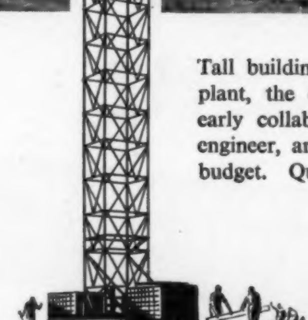

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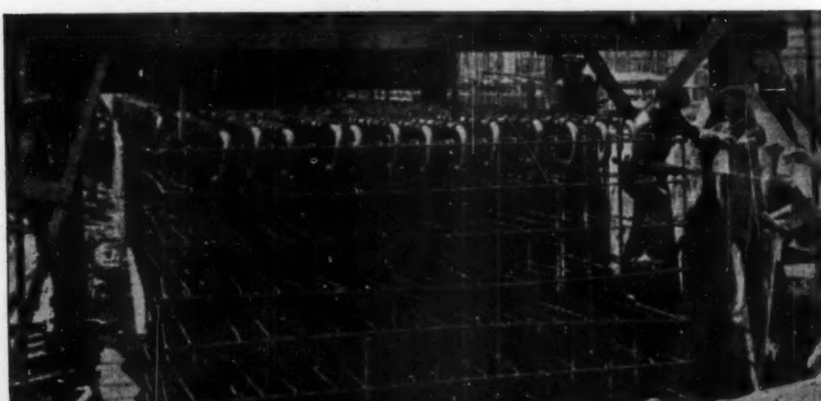

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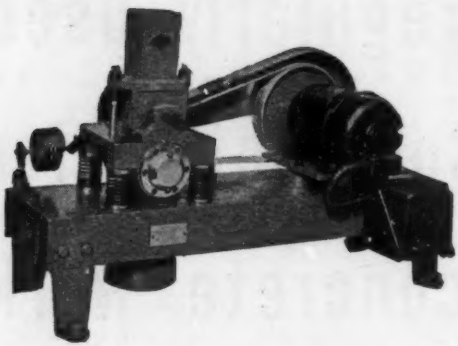

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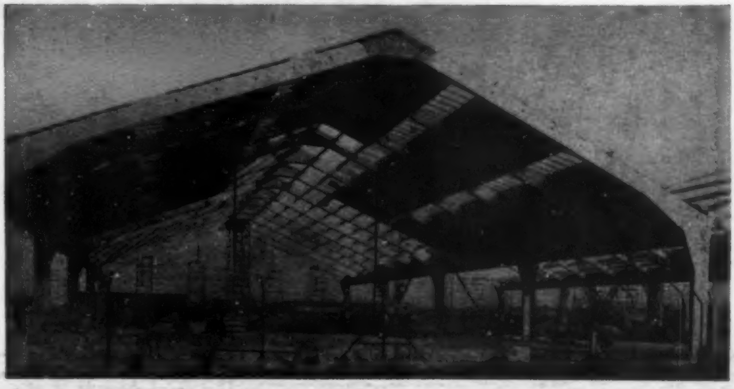

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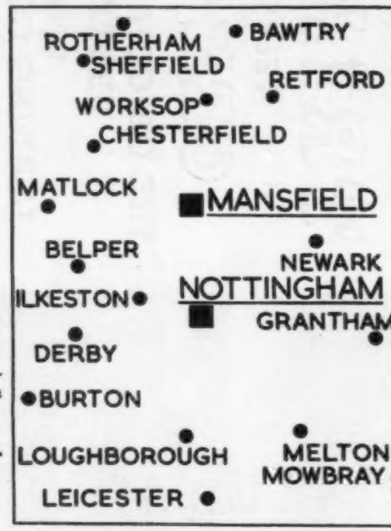

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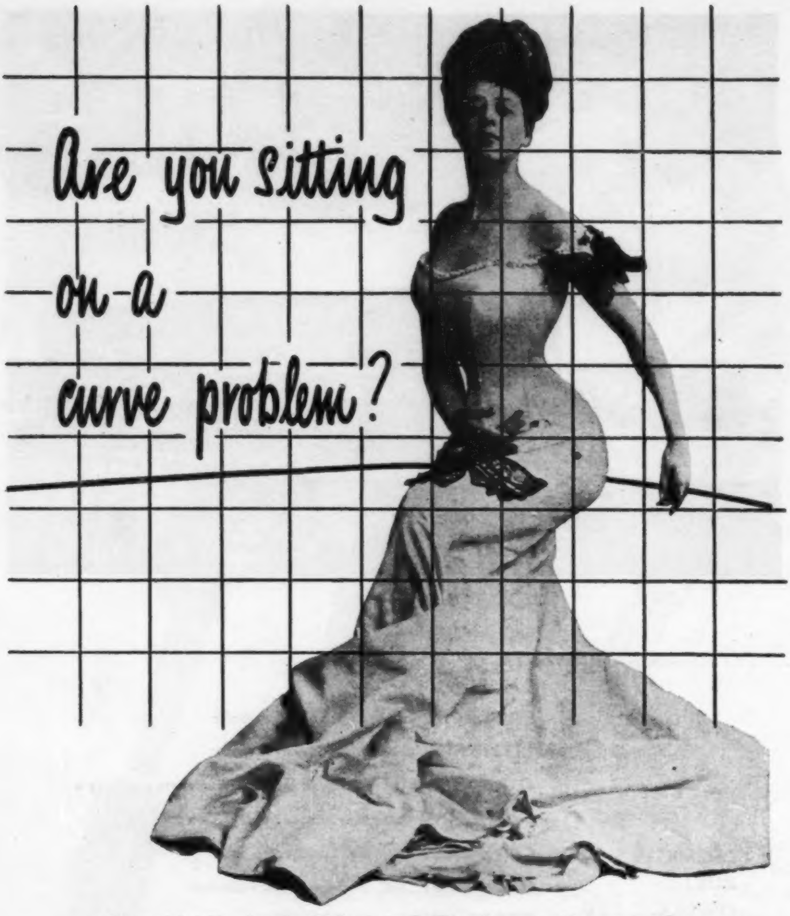

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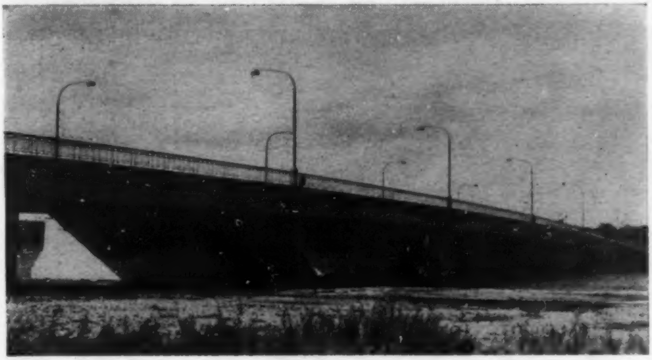

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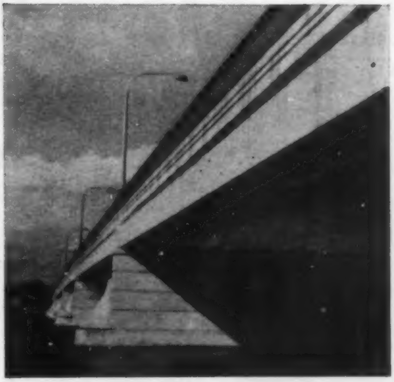

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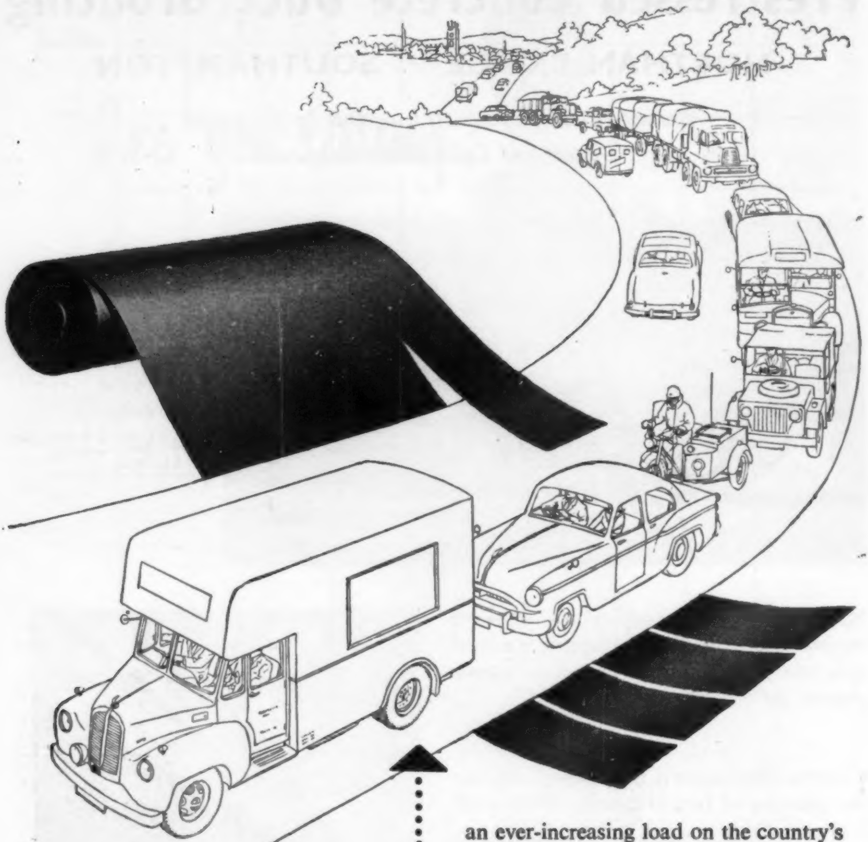

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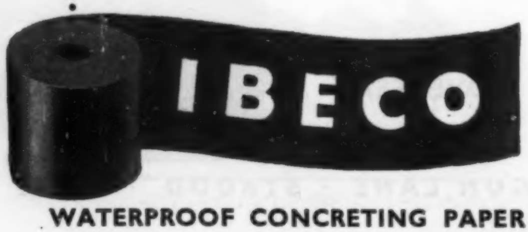

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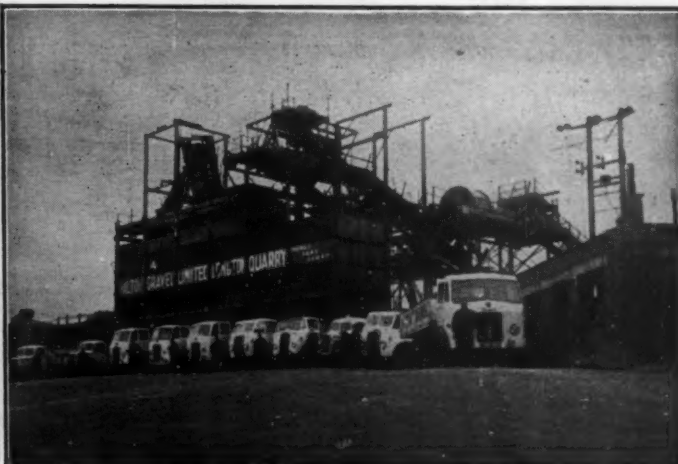

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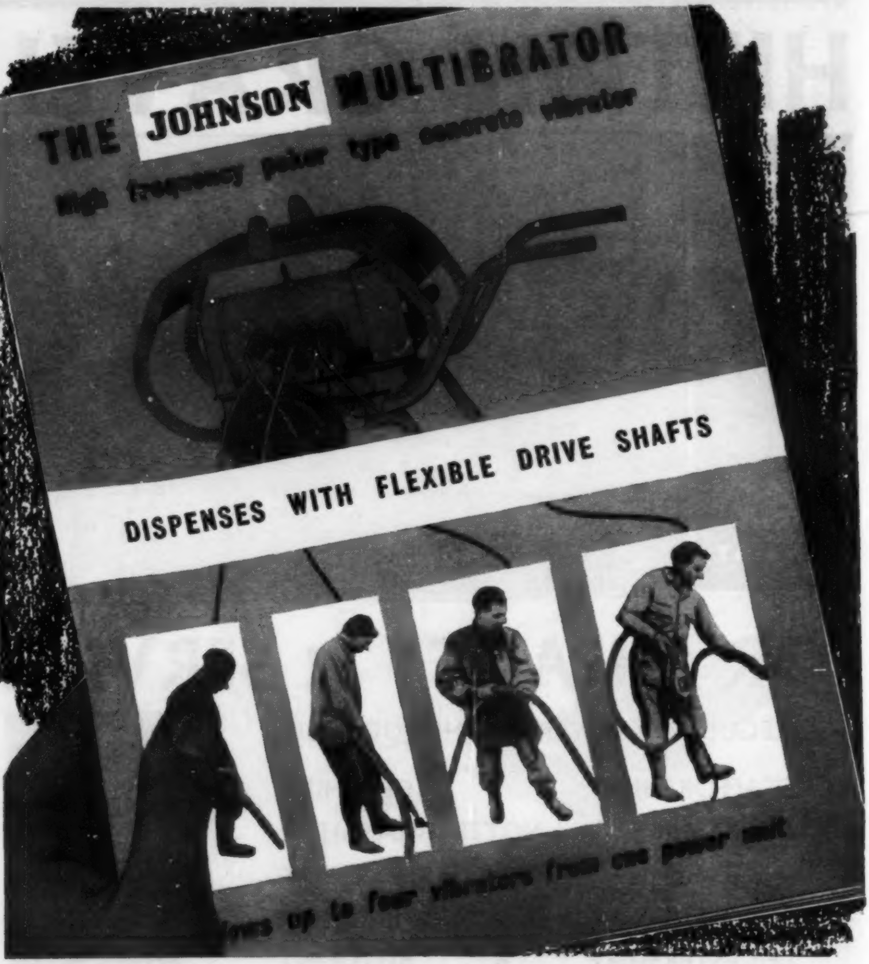

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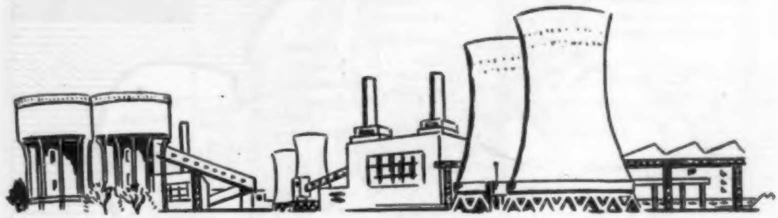

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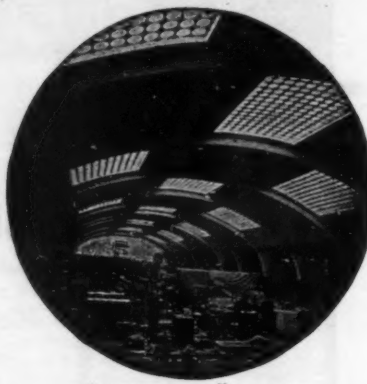

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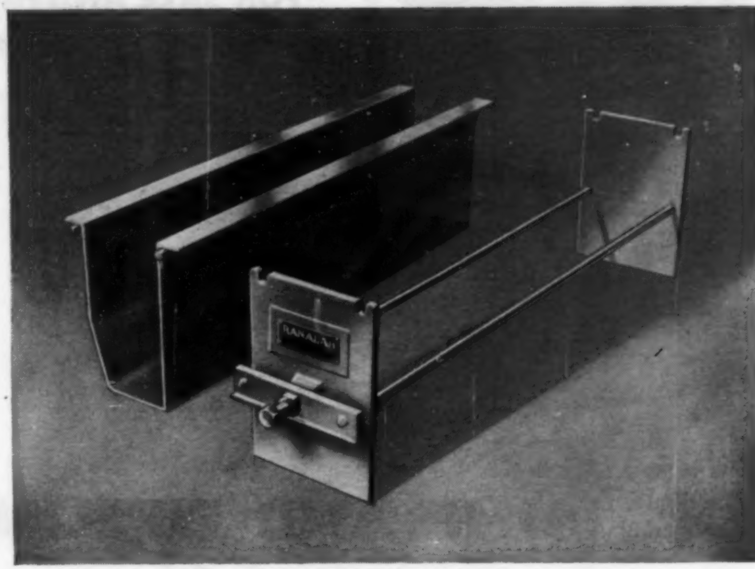

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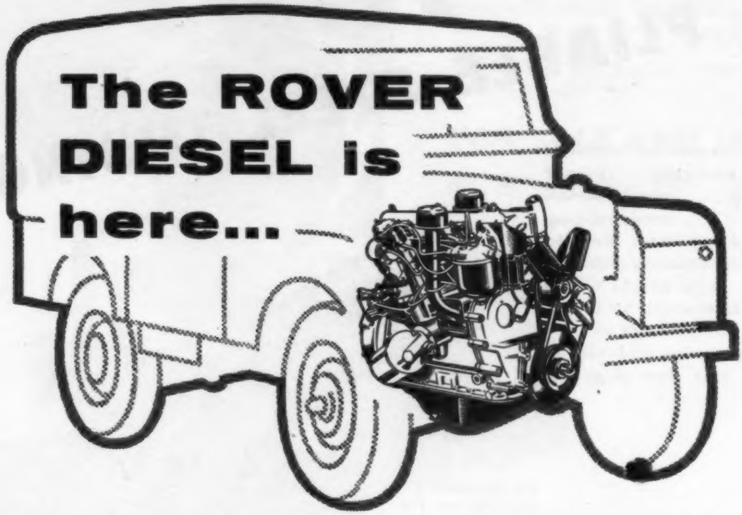

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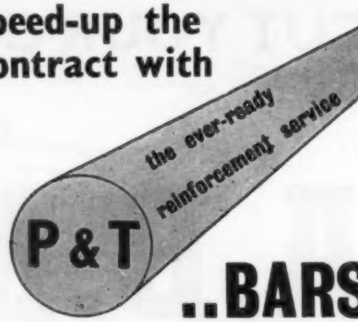

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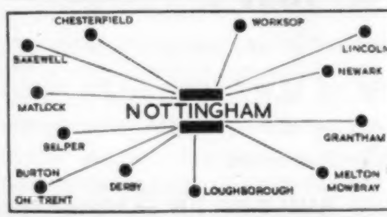

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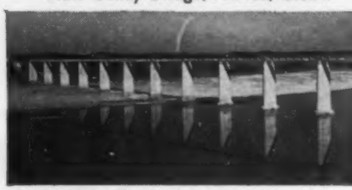

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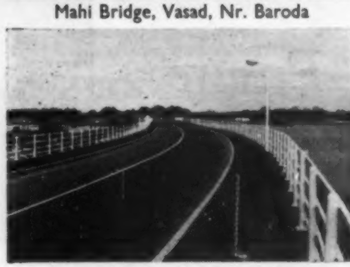

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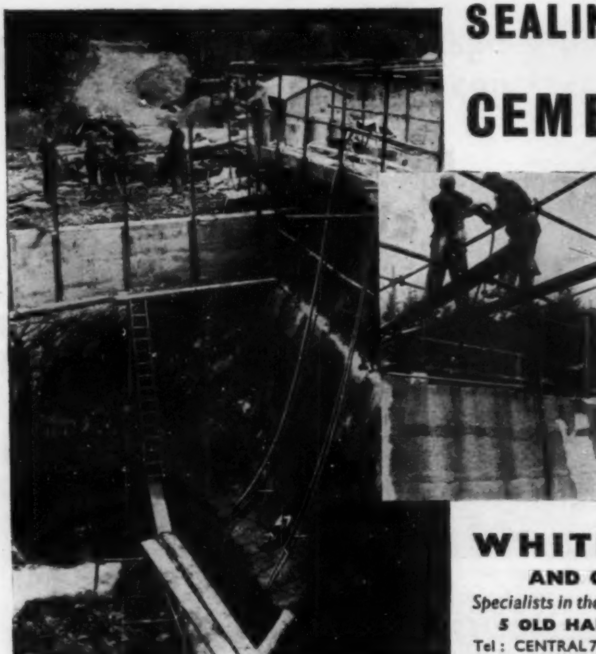

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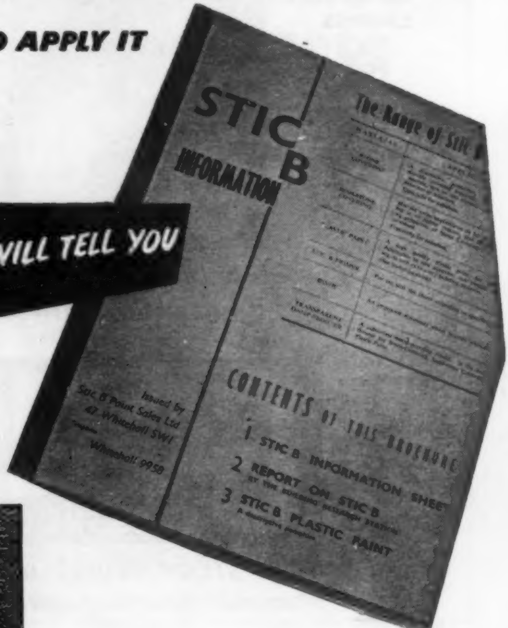

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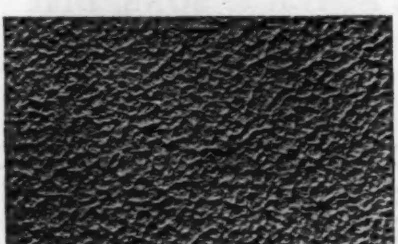

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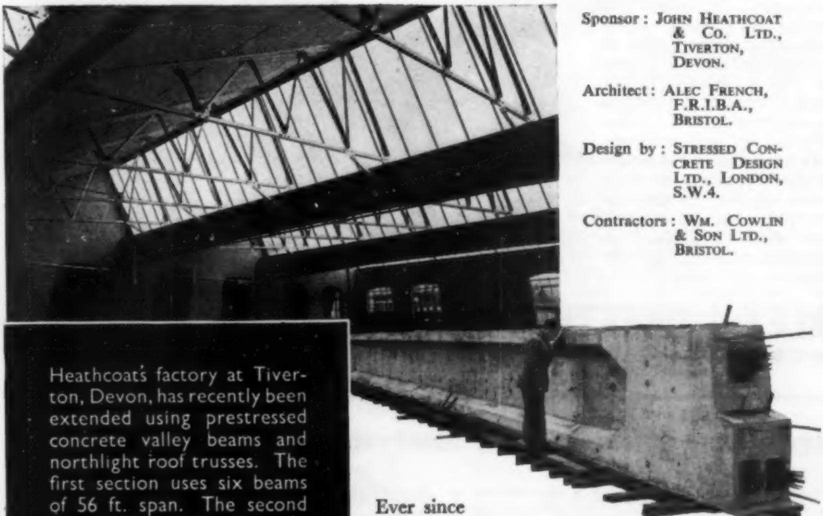

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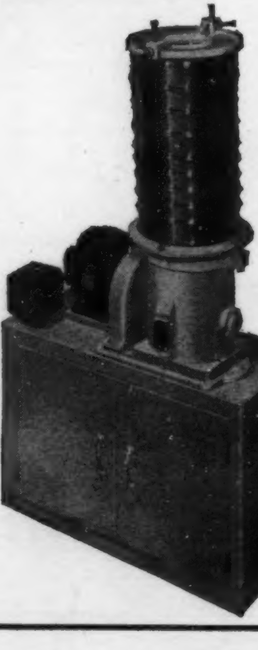

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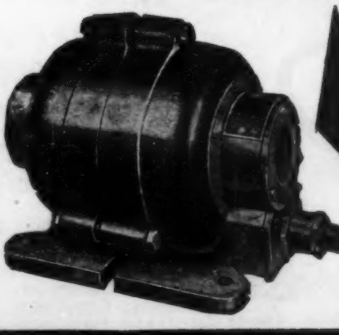

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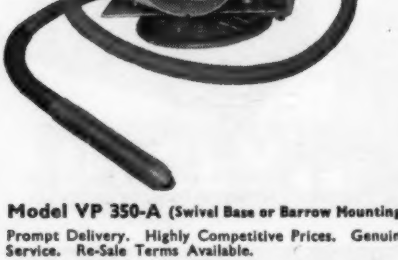

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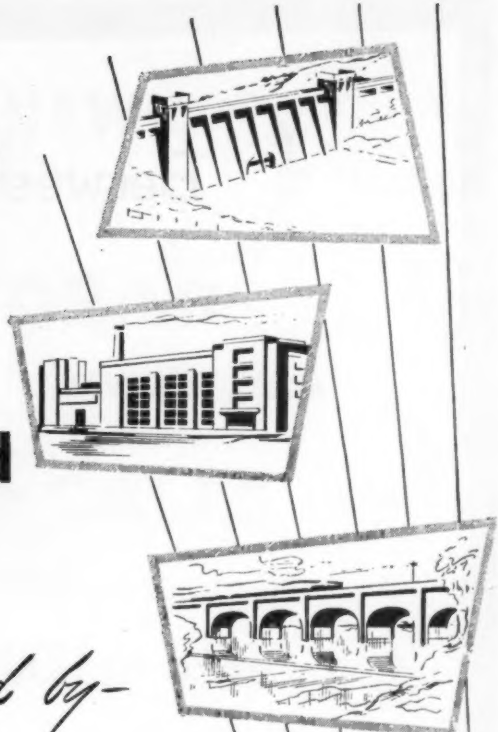

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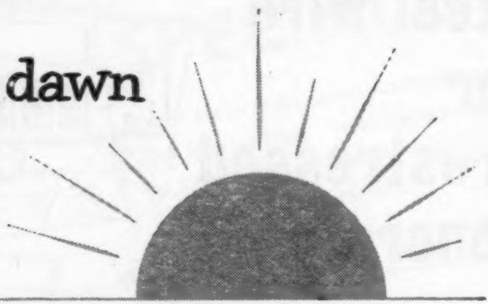

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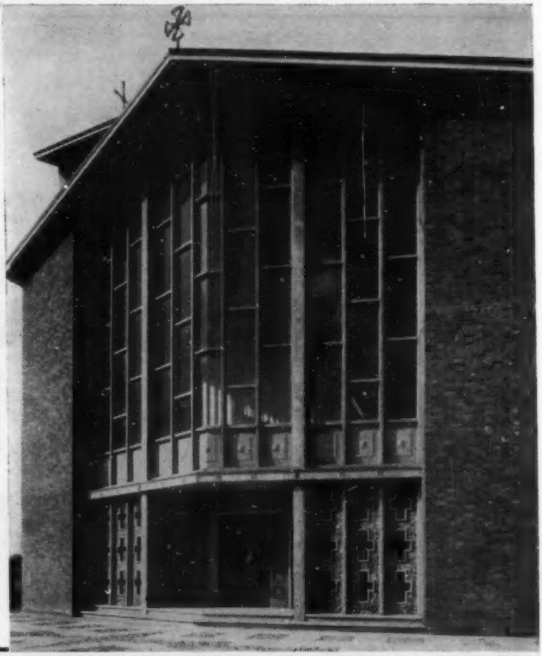

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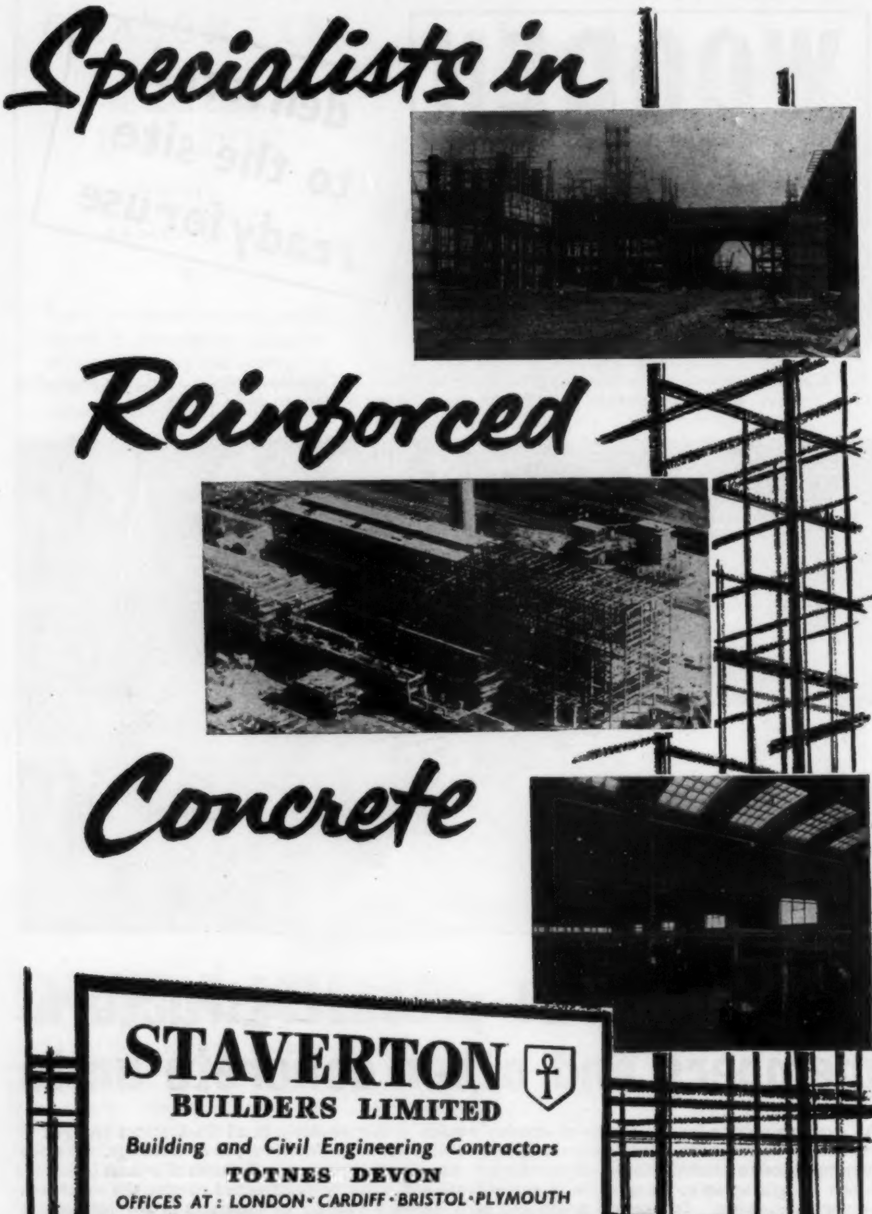
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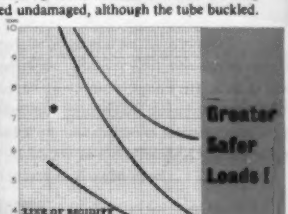

(Top curve) Ultimate Test curve for Self-Lock Prop with 5′ 6″ base tube and 8 swg scaffold tube inner.
(Centre curve) Typical strut failure curve 2″ scaffold tube with base and head plates.
(Bottom curve) Safe Load curve for Mills Self-Lock Prop

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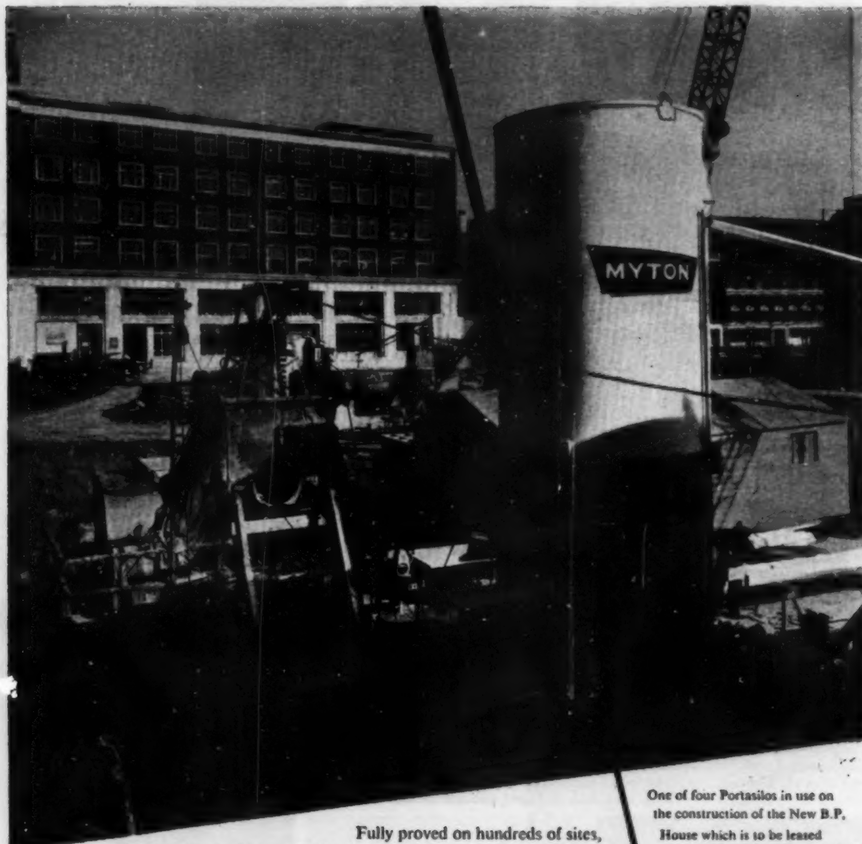

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Volume LII, No. 7. LONDON, JULY, 1957.

EDITORIAL NOTES

Technical Training in the U.S.S.R.

THE great advances in technology in the U.S.S.R. have led to deputations from other countries to study the training of technicians in that country. A recent deputation to visit the U.S.S.R. for this purpose comprised professors and lecturers of technical subjects at British universities and others concerned with technical training in this country, and a report on their investigation has now been published.* Major differences were found. For example, because of the scarcity of buildings, the courses are often worked two shifts a day, one from 8.30 a.m. to 2.15 p.m. and the other from 3 p.m. to 9 p.m. Evening classes are from 6 p.m. to 11 p.m. It is stated that the tests applied to students are less stringent than in Great Britain, and some of the Russian students who received diplomas would not have done so here. During his first two years at college a Russian student has some training in a workshop and time is given to foreign languages and general education, and during the last three years he spends about six months in industry. Only the fourth and fifth years are devoted to detailed study of a specialised branch of industry. At the Leningrad Polytechnic Institute only half of the full-time staff have higher degrees ; personal ability is the chief consideration in the appointment of staff. Teaching appointments are reviewed every five years. Some lectures are given to three hundred students at a time in halls where visibility for seeing demonstrations is poor. The proportion of women students varies from one-fifth in heavy technology such as power engineering to four-fifths or more in the lighter branches such as metallography. Women students in all branches of engineering comprise about half the total number of students, compared with about 1 per cent. in Great Britain. In the U.S.S.R. the average age of entry to the higher technical institutes is seventeen, the length of the course is five years or longer, the working week is of six days, there are thirty or more weeks of instruction each year, and the total hours of instruction in the course are between 4500 and 5100. It is seen that the teachers and students have much less spare time in the Soviet Union than in Great Britain, where instruction is given for only twenty-five to thirty hours a week during twenty-four weeks a year. In Soviet Russia technical students are exempt from military service.

The delegation was naturally handicapped by the short duration of the visit

* "Engineering Education in the Soviet Union." Obtainable from the Institution of Civil Engineers, the Institution of Mechanical Engineers, and the Institution of Electrical Engineers.

(nine days) and the language difficulty. Some of the report deals with matters that the members saw, but much of it repeats what they were told; the report states, however, that there is no reason to doubt its accuracy. The principal purpose of the delegation was to see what could be learned of the Russian educational system that could be applied in this country. In this it was not successful, due to the differences in the standards that were acceptable in each country and the difficulty of relating Russian standards to British. On the basis of a standard that would satisfy the examiners for associate membership of one of the major British engineering institutions, the number of men who reached this stage in Great Britain in the year 1954–1955 is said to be about five thousand. This can be compared only with the number awarded a diploma by an engineering institute in the U.S.S.R., and these are said to total 65,000 a year. Thus on the basis of the populations of the two countries the proportion of the people trained in engineering is about four times as great in Russia as in Great Britain. Whether the young British engineering graduates are better than the Russian the delegation is not able to say, but the report expresses some doubt on this point when in a badly-written sentence it states that "we would not be complacent about our young engineers even if they were greater than the Russian figures on a population basis".

The report is interesting but inconclusive, and its effect is spoilt by an endeavour to use it as propaganda for higher salaries for university staffs. It is stated that teaching is the highest-paid profession in the U.S.S.R., and that whereas a director of a teaching establishment is paid 7000 roubles a month a "director of a large enterprise" is paid only up to 6500 roubles a month. It is not, however, pointed out that in the U.S.S.R. a director of a large enterprise is not, as in this country, responsible to the shareholders for increasing business and dividends. The Russian director is a manager of a State-owned enterprise who is not responsible for getting business; also, he is paid a bonus based on costs. It is also stated that in the U.S.S.R. the teaching staffs can increase their incomes by undertaking research work for the government, whereas in Great Britain "some Government departments" do not allow university staffs to receive payment for research work done for these departments. In Russia those engaged in education are said to have "perquisites" in that motor-cars, living accommodation, and holiday camps are made available to them, whereas in the United Kingdom there are no "perquisites" for such people. It might, however, be mentioned that in Great Britain university staffs can and do buy a car or a house when they wish to do so. Also they can and do arrange their holidays to their liking in this or another country, and may consider this to be better than staying at a State-owned camp. It may further be pointed out that fees for consulting work and research may be, and are, received by university staffs in the United Kingdom from commercial firms, and that members of university staffs often arrange their lectures in book form, or write books, and receive royalties on their sale. When we compare conditions in Great Britain with those in the U.S.S.R., with its working week of six days, eight weeks' annual leave (no bank holidays), and the appointments "reviewed" every five years, it seems that our university staffs have little reason to envy the Russian director of a teaching establishment for receiving about the same salary as a works manager and "perquisites" that few British directors of teaching would wish to replace freedom of choice.

Reinforced Concrete Structures in Berlin.

A FEATURE of the International Building Exhibition being held in Berlin from July 6 to September 29 this year is an exhibition of models, photographs, and drawings of many of the more outstanding structures built in Berlin in recent years or now in course of construction. Some of these are described and illustrated in the following.

The new Berlin Congress Hall (*Figs.* 1 to 4) is being built by the Benjamin Franklin Foundation to the design of the American architect Mr. Hugh A. Stubbins, with whom two German architects, Messrs. Werner Duettmann and Franz Mocken, are co-operating.

Due to the marshy nature of the site, the building is founded on 500 precast piles 33 ft. long and with a bearing capacity of 50 tons each and 260 in-situ piles to carry 100 tons each. The building covers an area 330 ft. square. The main entrance leads into a two-story hall in the middle of the ground floor, around which are rooms and halls. The building also includes an exhibition hall of 10,500 sq. ft., a theatre with seating for 500, offices, a conference room to accommodate 200 people, a restaurant, and a post office, etc.

A platform will extend around the hall at first-floor level, supported on reinforced concrete columns. The main auditorium, with a floor area of about 12,500 sq. ft. and seating capacity for 1200 people, is supported on reinforced concrete columns.

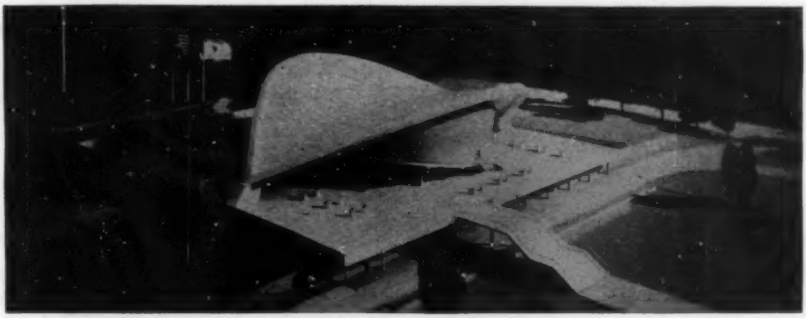


Fig. 1.—Congress Hall, Berlin.

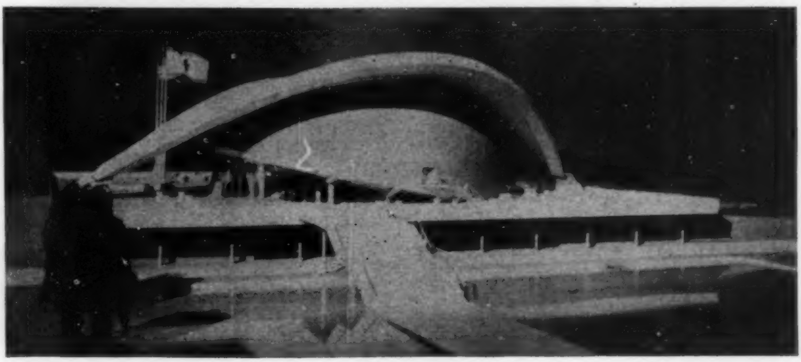


Fig. 2.—Congress Hall, Berlin.

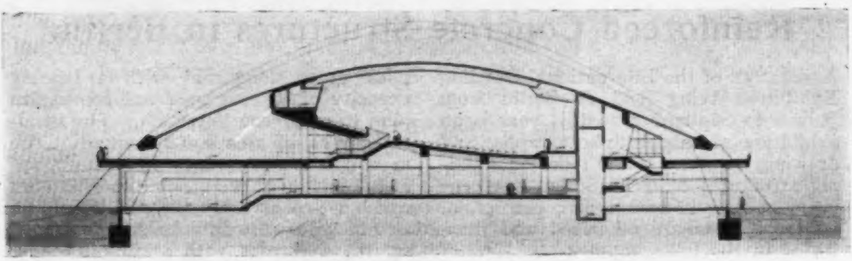


Fig. 3.—Longitudinal Section, Congress Hall, Berlin.

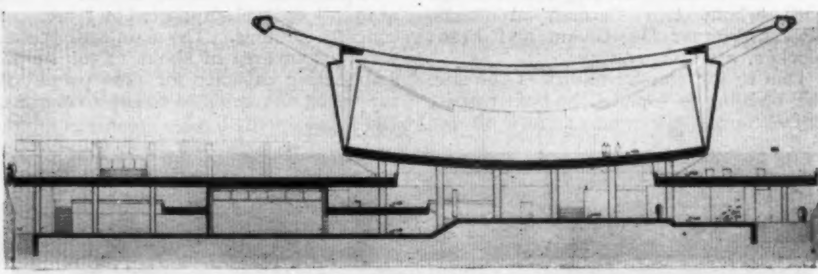


Fig. 4.—Cross Section, Congress Hall, Berlin.

The shape of the roof is intended to give the impression that it is floating over the building. Its shape is that of a saddle. It rests at the east and west on reinforced concrete beams about 10 ft. deep and 22 ft. wide. These are joined to two curved hollow beams, each 400 ft. long, between which the roof declines. The greatest height of the curved beams is 60 ft. The roof cantilevers 25 ft. beyond the beams, and is designed as a shell.

Fig. 5 shows an exhibition hall just completed at Berlin to the design of Mr. Bruno Grimmek. The hall has a floor area of more than 80,000 sq. ft. The structure is a composite steel and reinforced concrete construction. A mezzanine floor is suspended from the roof by high-tensile steel bars, so that the ground floor is free of columns. The frames, which span 165 ft., are of prestressed concrete, spaced 40 ft. apart,

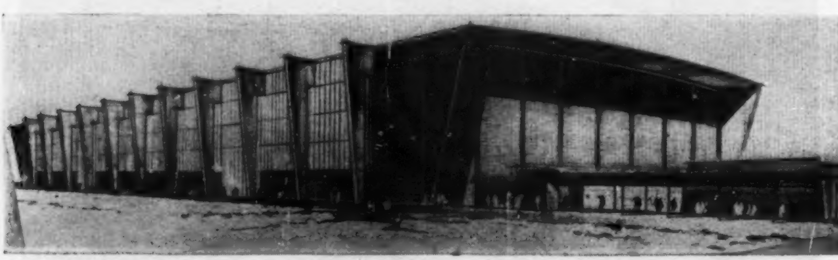


Fig. 5.—Exhibition Hall, Berlin.

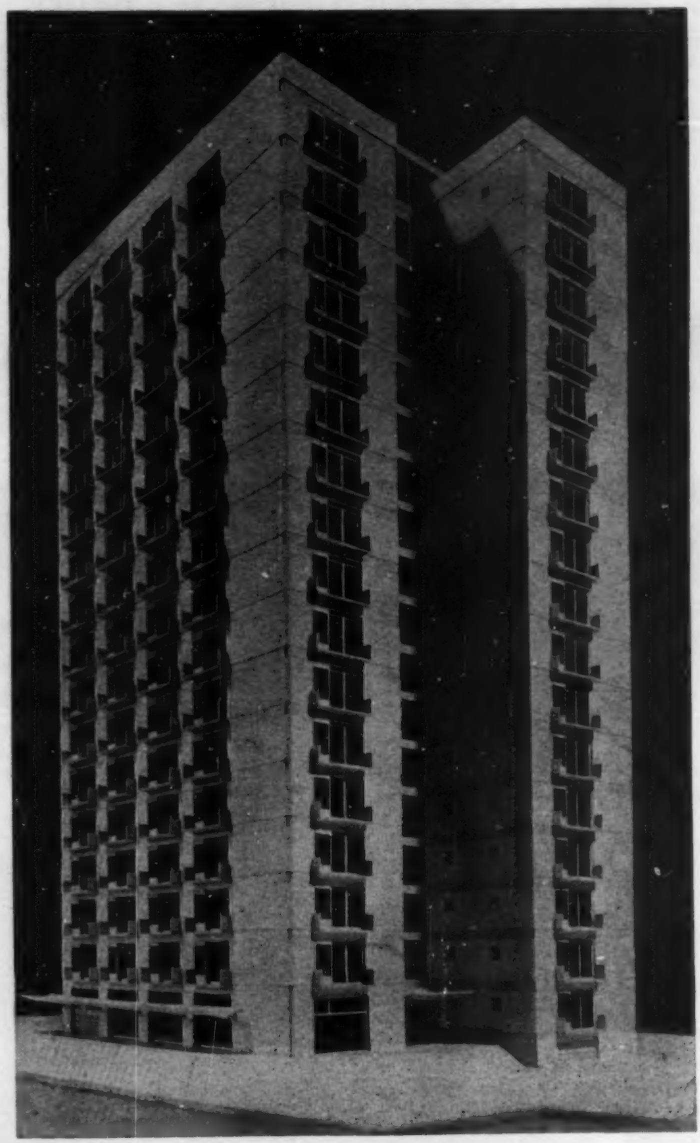


Fig. 6.—Residential Flats, Berlin.

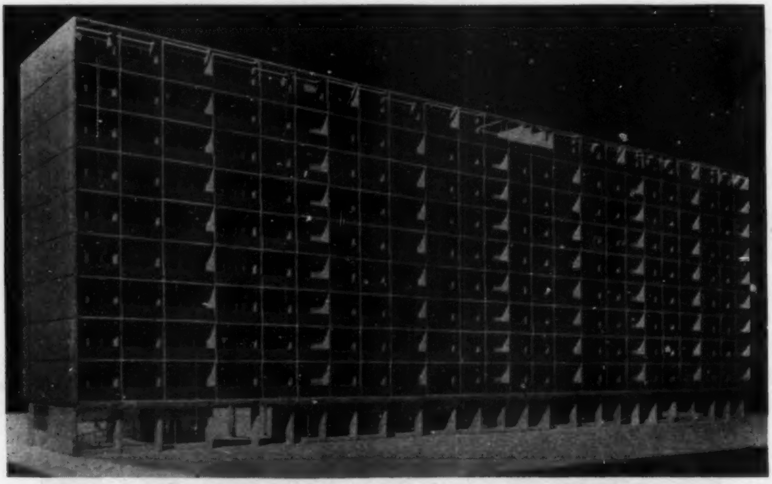


Fig. 7.—Residential Flats, Berlin : Front Elevation.

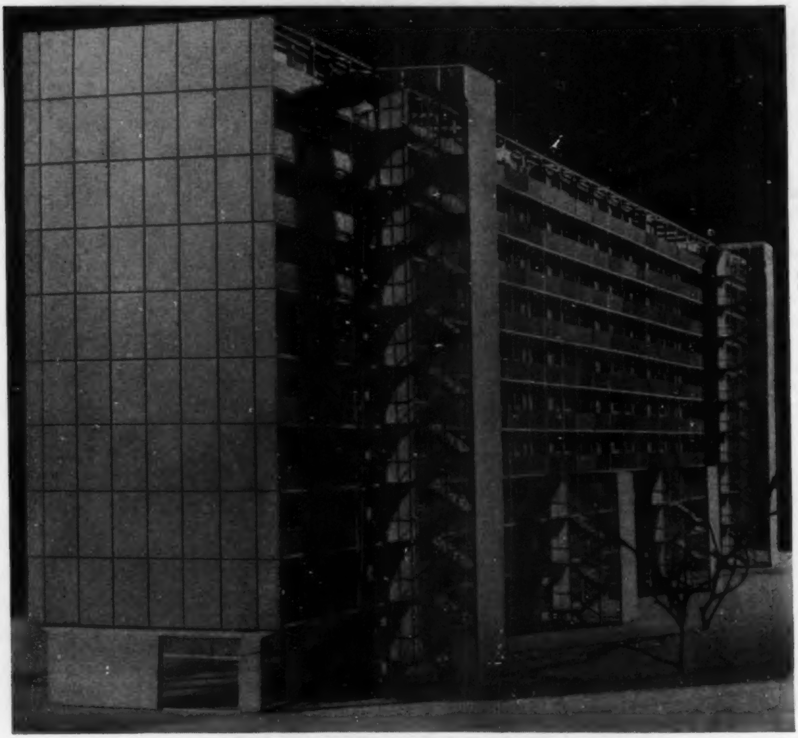


Fig. 8.—Residential Flats, Berlin : Rear Elevation.

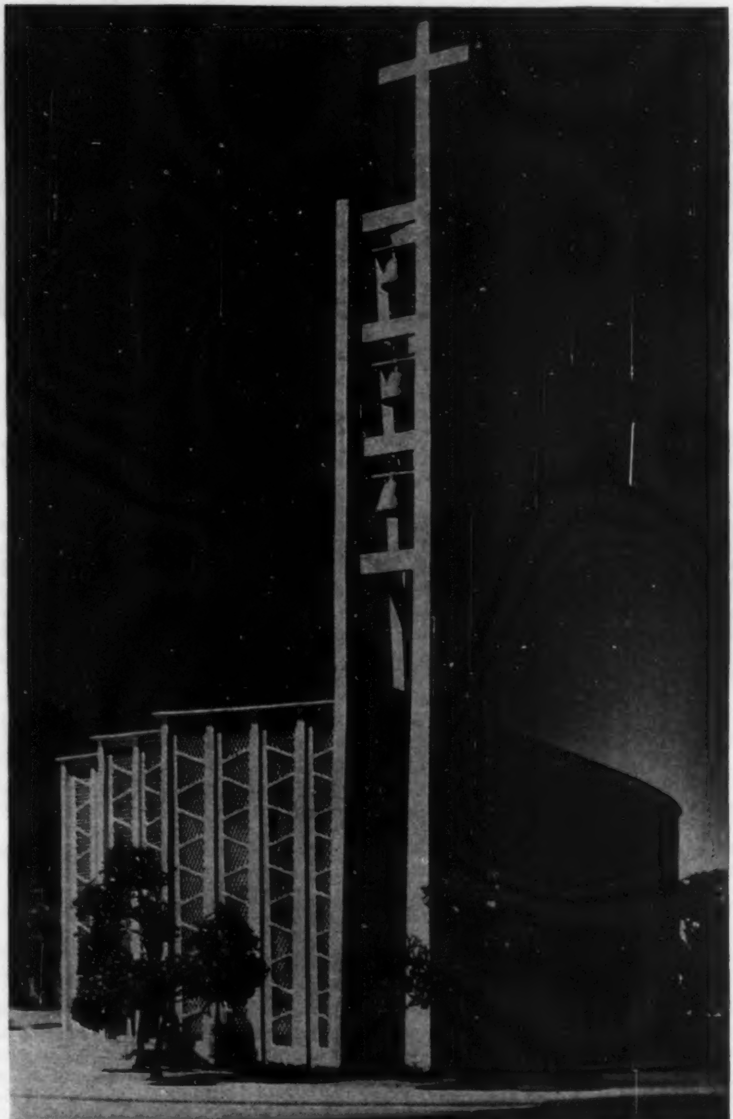


Fig. 9.—Roman Catholic Church, Berlin.

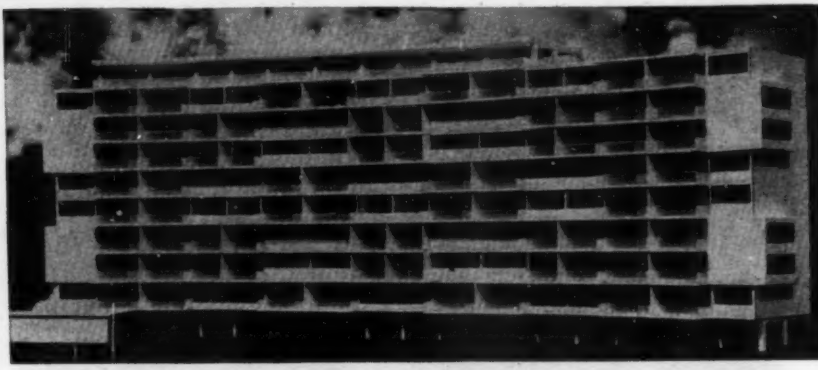


Fig. 10.—Residential Flats, Berlin.

and the precast reinforced concrete purlins are 6 ft. 6 in. apart. The roof slab is of aerated concrete 3 in. thick. The total height of the frames is 61 ft.

The residential flats shown in *Fig.* 6 are seventeen stories high and cover an area of nearly 5000 sq. ft. The architects are Professor Klaus Mueller-Rehm and Mr. Gerhard Siegmann. The walls are of reinforced concrete cast in place, and the outer walls are faced with precast concrete slabs of story height including a layer of insulating material.

The residential flats shown in *Figs.* 7 and 8 (architects Messrs. Fritz Jaenicke and Sten Samuelson of Malmö, Sweden) will be 281 ft. long by 33 ft. wide by 100 ft. high and will contain 68 flats. The building will be a reinforced concrete framed structure with walls of lightweight precast slabs.

The church shown in *Fig.* 9 is being built in the Berlin Hansa district to the design of Professor Willy Kreuer. The plan of the nave is similar to a parabola. One side of the curve, nearest to a railway, is an unbroken wall so that services will not be disturbed by railway traffic. The other sides have large areas of glass. The church will be of reinforced concrete. The plain wall is to be painted on the inside, with the colour becoming lighter towards the altar. The spaces between the columns of the other walls will be filled with precast concrete and glass.

Residential flats designed by Professor Walter Gropius and Professor Wils Ebert and to be built in Berlin are shown in *Fig.* 10. The structure will be about 82 ft. high, 33 ft. wide, and 275 ft. long. The building is to be in framed reinforced concrete. The outer walls will be in white concrete, and the fronts of the balconies will be covered with white enamelled steel. The window frames are to be of aluminium.

The Volume of Construction.

According to the Ministry of Works, the total value of building and civil engineering work done during the first quarter of the year 1957 was £525,000,000, compared with £474,000,000 in the first quarter of the previous year. The number of men employed in these industries was 1,060,000 in the first quarter of 1957 compared with 1,073,000 in the first quarter of 1956. The index number of the cost of new construction increased from 134 in the first quarter of 1956 to 139 in the first quarter of 1957. The value of work done by direct labour by Government departments, local authorities and nationalised industries was £83,000,000 in the first quarter of 1956 and £87,000,000 in the first quarter of 1957.

Estimating the Size of Rectangular Sections.

By F. H. TURNER, B.Sc., A.M.I.C.E., A.M.I.Struct.E., D.I.C.

FOR preliminary designs and estimates a rapid but approximate method of determining the size of sections is frequently required. It is assumed that the span, total load, maximum bending moment, and maximum shearing force have already been evaluated, that the section is to have not less than the "economical" depth, and that no reinforcement to resist shearing forces is desired. The following notation is used: d, effective depth; b, breadth; l, span; w, uniform load per foot; W, total load; S, maximum shearing force; M, maximum bending moment; A_s, area of tensile reinforcement required; $k = \frac{M}{Wl}$; $k_1 = \frac{S}{W}$; $k_2 = \frac{d}{l}$; $k_3 = \frac{b}{l}$.

Since d^2 must be equal to or greater than $\frac{M}{Qb}$, and $\frac{S}{a_1bd} = 100$,

$$k_2^2l^2 > \frac{kWl}{200k_3l} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and

$$\frac{k_1W}{a_1k_2k_3l^2} = 100 \quad . \quad . \quad . \quad . \quad . \quad (2)$$

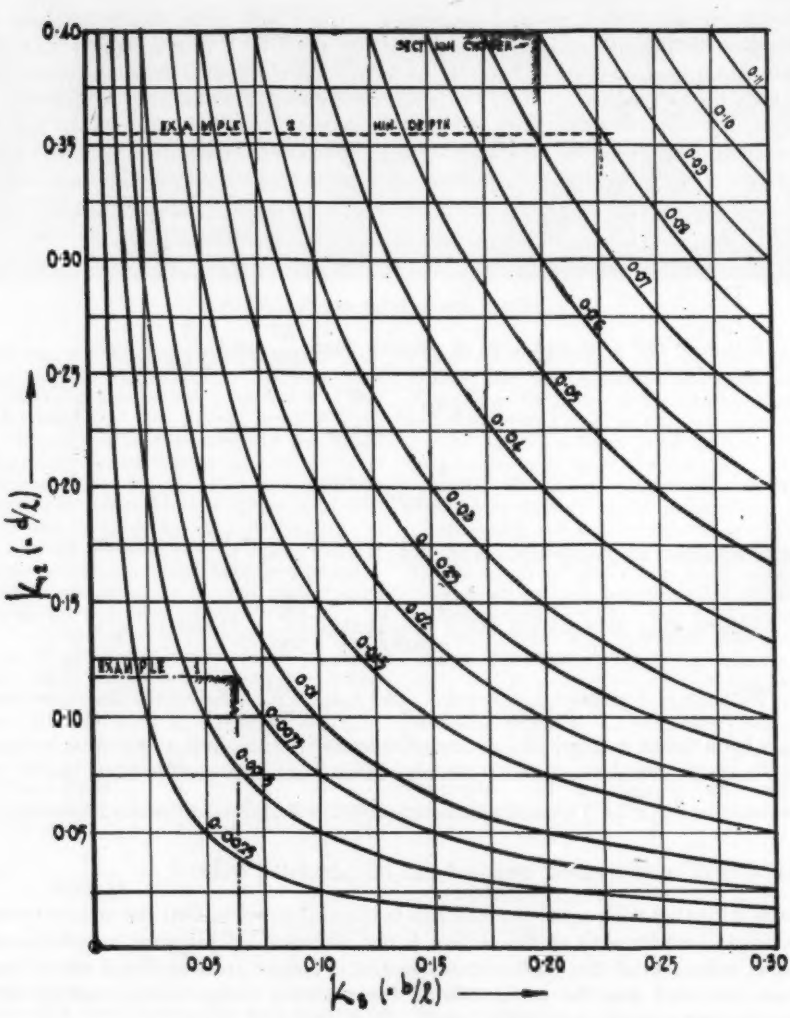
From equation (2), since $a_1 = 0{\cdot}85$, $k_2k_3 = \frac{k_1W}{100a_1l^2} = \frac{k_1W}{85l^2}$.

Substituting in equation (1), $\left(\frac{k_1W}{85l^2}\right)k_2l^2 > \frac{kW}{200}$. Therefore $k_2 > \frac{0{\cdot}425k}{k_1}$.

METHOD.—Evaluate $k_2 = 0{\cdot}425\,\frac{k}{k_1}$, and $k_2k_3 = \frac{k_1W}{85l^2}$. Select the line representing k_2k_3 on *Graph* I. Any pair of co-ordinates to this line indicates a cross section with sufficient resistance to the shearing force. Select a section such that k_2 is greater than $\frac{0{\cdot}425k}{k_1}$. This indicates a section that will also resist the bending stresses. The area of reinforcement required can be calculated from $A_s = \frac{M}{15{,}500d}$. The graph is plotted with equal vertical and horizontal scales so that the ratio between the depth and breadth of the section is not distorted. This gives an additional visual indication of the most suitable section. Larger cross-sectional areas than those indicated may be used, making the shearing stress correspondingly less than the maximum permissible stress.

EXAMPLE I.—A beam spans 15 ft. ($l = 180$ in.), the total load is 30,000 lb., the maximum bending moment is 1,000,000 in.-lb., and the maximum shearing force is 20,000 lb.

$$k_1 = \frac{2}{3} = 0{\cdot}67\,;\; k = \frac{1{,}000{,}000}{30{,}000 \times 180} = 0{\cdot}185. \quad \frac{0{\cdot}425k}{k_1} = \frac{0{\cdot}425 \times 0{\cdot}185}{0{\cdot}67} = 0{\cdot}1175.$$



Graph No. 1.—Curves Corresponding to Various Values of k_1k_2.

$$k_2k_3 = \frac{k_1W}{85l^2} = \frac{0{\cdot}67 \times 30{,}000}{85 \times 180^2} = 0{\cdot}00727.$$

Mark the minimum depth (0·1175) on the graph. The corresponding breadth is 0·063, which gives a reasonable section. Adopting these sizes,

$$d = 0{\cdot}1175 \times 180 = 21{\cdot}1 \text{ in. and } b = 0{\cdot}063 \times 180 = 11{\cdot}4 \text{ in.}$$

Make the beam 2 ft. by 1 ft., and $A_s = \dfrac{1{,}000{,}000}{15{,}500 \times 21{\cdot}1} = 3{\cdot}06$ sq. in.

EXAMPLE II.—A cantilevered beam projects 6 ft. ($l = 72$ in.), the load is 35,000 lb., and the maximum bending moment is 2,100,000 in.-lb.

$$k_1 = 1. \quad k = \frac{2{,}100{,}000}{35{,}000 \times 72} = 0{\cdot}833. \quad 0{\cdot}425\frac{k}{k_1} = 0{\cdot}425 \times 0{\cdot}833 = 0{\cdot}355.$$

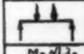
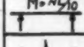
$$k_2k_3 = \frac{k_1W}{85l^2} = \frac{35{,}000}{85 \times 72^2} = 0{\cdot}0795.$$

From the graph, $k_2 = 0{\cdot}4$ and $k_3 = 0{\cdot}2$ give a reasonable section. Therefore $d = 0{\cdot}4 \times 72 = 28{\cdot}8$ in., and $b = 0{\cdot}2 \times 72 = 14{\cdot}4$ in. Make the beam 2 ft. 6 in. by 1 ft. 3 in., and $A_s = \dfrac{2{,}100{,}000}{15{,}500 \times 28{\cdot}8} = 4{\cdot}7$ sq. in.

The various constants are shown in *Table* I for the common conditions of loading. If several beams are to be estimated a table can be made under the headings k_1, k, $\dfrac{0{\cdot}425k}{k_1}$, $\dfrac{k_1}{85l^2}$, $d = k_2l$, $b = k_3l$, and $A_s = \dfrac{M}{15{,}500d}$.

The method can be adapted to obtain approximate dimensions for tee-beams by evaluating k_2k_3, where $k_3 = \dfrac{\text{breadth of rib}}{\text{span}}$, and determining the minimum value of k_2 corresponding with the known ratio $k_3 = \dfrac{\text{breadth of flange}}{\text{span}}$.

TABLE I.

LOADING	k_1	k	$0{\cdot}425\,k/k_1$	$k_1/85$
	0·5	0·25	0·213	0·00588
	0·5	0·125	0·106	0·00588
	0·5	0·1$\dot{6}$	0·142	0·00588
	0·5	0·10	0·085	0·00588
	0·5	0·125	0·106	0·00588
	0·5	0·083$\dot{3}$	0·071	0·00588
	1·0	1·0	0·425	0·01175
	1·0	0·5	0·213	0·01175

EXAMPLE III.—A continuous tee-beam spans 25 ft. between supports. The bending moment at a support is 2,500,000 in.-lb. and at midspan 2,000,000 in.-lb. The total load is 100,000 lb. and the maximum shearing force 60,000 lb. The breadth of the flange is 60 in.

Rectangular section at the supports:

$$k_1 = 0{\cdot}6,\ k = \frac{2{,}600{,}000}{100{,}000 \times 300} = 0{\cdot}083\,;\ \ 0{\cdot}425\frac{k}{k_1} = \frac{0{\cdot}425 \times 0{\cdot}083}{0{\cdot}6} = 0{\cdot}0059,$$

$$\frac{k_1 W}{85 l^2} = \frac{0{\cdot}6 \times 100{,}000}{85 \times 300^2} = 0{\cdot}0078.$$

From the graph it is seen that a very wide section is required to resist bending. Adopting a section such that $k = 0{\cdot}15$ and $k_3 = 0{\cdot}065$, $d = 0{\cdot}15 \times 300 = 45$ in. and $b = 0{\cdot}065 \times 300 = 19{\cdot}5$ in. Use a section 48 in. by 21 in.

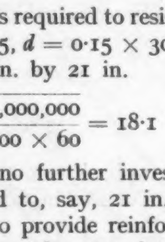
$$\text{Tee section at midspan: } d_{min.} = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{2{,}000{,}000}{200 \times 60}} = 18{\cdot}1 \text{ in.}$$

Since the shearing stress at midspan is small no further investigation is required and the depth at midspan may be reduced to, say, 21 in.

In the design of continuous beams it is usual to provide reinforcement to resist the shearing forces. This can be taken into account by assuming a constant nominal shearing stress and decreasing the calculated values of $k_2 k_3$ in the ratio $\dfrac{100}{\text{“ nominal ” shearing stress}}$.

Adjusting the Level of Foundations.

A METHOD of adjusting the level of the foundations of a building which may be subjected to variable settlement has been devised by the French engineer M. Albert Caquot and is being used in Le Havre in the " Porte Océan ", a building of fourteen stories, where a variable settlement of 2½ in. is expected. The accompanying cross section through a column and foundation illustrates the method. The lower part of the column is reduced in size so that it may pass through a ring-beam anchored to the foundation. This ring-beam contains a bed of sand laid on top of the foundation, on which the precast base of the column is placed. Extending from the top of the precast base are reinforcement bars placed so as to lap with the longitudinal reinforcement of the remainder of the column to be cast in place. A tube passes through the ring-beam to the bed of sand and is sealed by a removable cap at its outer end. When it is necessary to alter the level of the base



of the column the cap is removed and an auger is inserted into the sand so that by rotating the auger some of the sand is removed and the base of the column slides down within the ring-beam. Another tube passes through the precast base to the sand so that it is possible to inject cement-grout under pressure to raise a column or to stabilize the sand at any desired level. The foregoing is abstracted from a recent number of the Belgian journal " La Technique des Travaux ".

Design of Reinforced and Prestressed Bridges.

By DR. RICCARDO MORANDI.

The following is an abstract of a lecture entitled "Modern Conceptions in the Planning of Bridges in Reinforced Concrete and in Prestressed Concrete", read by Dr. Riccardo Morandi, the well-known Italian engineer, at a meeting in London organised by the Joint Committee on Structural Concrete under the auspices of the Cement and Concrete Association. The works described were all designed by the lecturer.

Arch Bridges.

For the reconstruction of the San Niccola bridge over the river Arno in Florence, destroyed in the war, a single arch with a span of 290 ft. and a rise of 26 ft. was used. For this bridge it seemed opportune to abandon completely the concept of an arch with stiffening slabs and to use a curved beam equally flexible at the crown and the springings, in which the deck would be supported on the arch by vertical slabs disposed so as not to offer any resistance to the deformation of the arch due either to its own weight or to live loads. The arch (*Figs.* 1 and 2) is a cellular structure of a total depth of 3 ft. 6 in. at the crown and 11 ft. 6 in. at the springings. The bottom slab is 1 ft. thick throughout most of its length, and near the springing there are vertical slabs at about 10 ft. centres. The top slab has a constant thickness of 1 ft.

Fig. 1.—San Niccola Bridge, Florence.

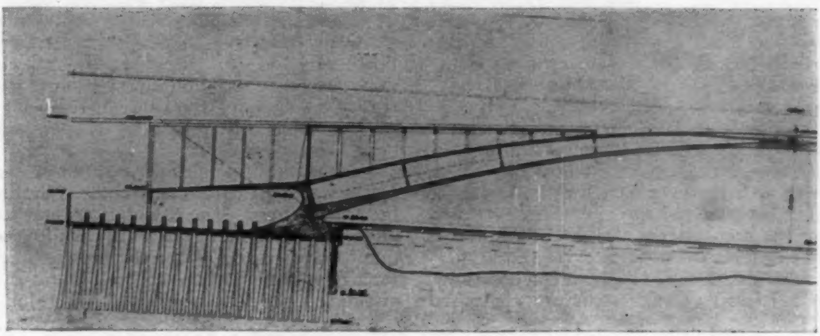


Fig. 2.—San Niccola Bridge, Florence: Half-longitudinal Section.

The cellular structure projects beyond the springings for about 112 ft. It is supported on inclined hollow piles and filled with sand and gravel; this contains the resultant of the forces at the level of the foundation within an angle of 32 deg. to the vertical. Excluding the foundations,

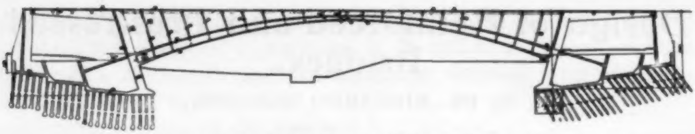


Fig. 3.—Paguita Bridge, Caracas.

3·28 cu. ft. of concrete and 24·8 lb. of mild steel were used per square foot of deck. The behaviour of the bridge when the centering was removed, during tests, and in use showed that the modulus of elasticity is $4 \cdot 97 \times 10^6$ lb. per square inch.

The Paguita bridge (*Fig.* 3) in Caracas, Venezuela, is a shallow arch spanning 295 ft. and rising barely 23 ft. The thickness of the arch, especially at the abutments, had to be kept as small as possible, and the bridge is at an acute angle of skew ; also the horizontal thrust of the arch had to be as little as possible because of the nature of the soil. Prestressing was of great help in solving the problems, and it was possible to experiment with the concept of self-regulation of the funicular polygon by applying diagonal forces along the arch. An arch with such a small rise produces a thrust of considerable magnitude on the foundations, and because the abutments rested on soils of completely different physical characteristics the piled foundations had to be different. The structures above the piles were prestressed in order to resist, with as little deformation as possible, the high bending and shearing stresses caused by the non-convergence of the resultants of the forces transmitted by the arch.

The arch, which is at an acute skew, is a curved cellular structure composed of upper and lower slabs and transverse slabs between the arch and the deck. At the crown the structure is about 3 ft. deep and at the springings about 8 ft. The design of such a thin vault with a small rise was much influenced by the difference in rigidity at the crown and the springings. It was believed that the skew would cause the vault to act differently from the theoretical behaviour. For example, the difference between the moment of inertia of a section normal to the lower slab and that of a section normal to the upper slab is much greater at the springings than at the crown. For this reason the arch was divided into a number of independent beams tied only by transverse slabs whose function is to cause all the girders to deform identically at given distances from each springing. The calculations for the vault made allowance for the oblique forces transmitted by the deck supports, which produce forces in the arch counteracting its thrust, so changing the value of the bending moments and producing economical design.

Near the abutments the resultant of the forces was kept within the required limits by prestressing. The calculations had to allow for the effect of the additional forces due to prestressing, since the prestressing force does not act at the centres of gravity of the sections, and for a further reduc-

Fig. 4.—Lupara Bridge, Genoa.

tion of thrust resulting from the axial contraction produced by the pre-tensioned cables. As a result the quantities of concrete and steel per square foot of deck were reduced from 3·3 cu. ft. and 26·5 lb. per square foot for the San Niccola bridge to 2·64 cu. ft. and 18·2 lb. per square foot for the Caracas bridge in spite of the reduction in the rise of the arch.

The advantages achieved in the last example apply to any arch. An arch is economical if the bending moments are similar throughout its span; a great difference in thickness at the crown and at the springings should always be avoided. An arch has been designed with the thickness at the crown greater than that at the abutments. This is not economical, and the modern tendency is to make the thickness at the springings slightly greater than at the crown. In all arch bridges the roadway consists of a system of beams of multiple spans more or less horizontal, and prestressing may be used to lighten the structure. In the construction of very large bridges, the elastic behaviour of the piers must be considered. The most economical results can be obtained by using isolated supports which taper and have their greatest moment of inertia at the road and their least at the arch, thus causing little distortion of the arch.

A comparison between the amounts of concrete and reinforcement required for this type of arch and those required for an arch designed by the elastic method, originally devised by Maillard, is of interest. The Lupara bridge (*Fig.* 4, designed by Dr. Morandi) and the Caifa bridge, of the Maillard type, are similar and both have a span of 390 ft. In the Lupara bridge, 3·15 cu. ft. of concrete and 22·5 lb. of steel were used per square foot of deck; in the Caifa bridge, 4·35 cu. ft. of concrete and 28·5 lb. of steel were used.

In Italy, tubular steel centering of the Innocenti type (*Fig.* 5) is frequently used for the construction of bridges with long spans because it is light and can be erected and dismantled rapidly. It is sufficiently rigid to reduce considerably the effects of distortion owing to the deformation of the centering. For the Lupara bridge 200 tons of steel centering were used. It is desirable that the centering should be removed gradually so as to develop as slowly as possible the stresses in the arch due to its own weight.

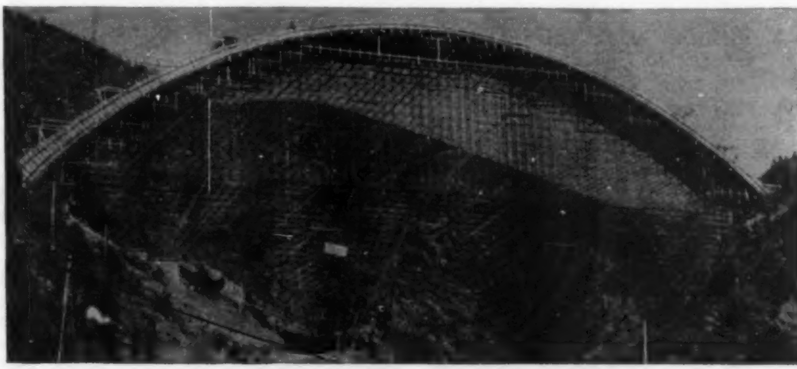


Fig. 5.—Steel Centering (Innocenti type).

At present, the Cruciani type of timber shuttering (*Fig.* 6) is being much used, even for arches spanning more than 300 ft.

Bridges with Horizontal Beams.

For bridges with straight beams of reinforced concrete the internal stresses due to the live load are almost always small compared with those due to the dead load. Attempts have been made to reduce the dead load, and it is now possible to build beams with high-strength concrete and ordinary reinforcement for spans of some 160 ft. with the use of about 1·65 cu. ft. of concrete per square foot of deck. For a bridge in reinforced concrete the ratio of dead load to live

Fig. 6.—Wooden Centering (Cruciani type).

load per unit of surface is about one; for a prestressed concrete structure it is about 0·6 to 0·7. This ratio may be reduced by further developments, but eventually the effects of excessive deformation will have to be considered.

Although it is generally believed that prestressed concrete is not as suitable as reinforced concrete for simply-supported beams of small span, bridges of simply-supported beams have been built in which prestressing has been used to obtain lightness and economy for spans of 70 ft., using a series of prestressed concrete beams on existing supports. Each beam comprises five webs of ovoidal section tied by the upper slabs and by transverse members. The profile of the sections is made economically possible by casting the various elements in quantity, using steel shuttering and the vacuum process. The design made the maximum use of the material employed, although the maximum stresses were similar to those adopted for other prestressed structures. For each square foot of deck, 0·96 cu. ft. of concrete, 3·4 lb. of high tensile steel, and 3·7 lb. of mild steel were used. Because of their light weight the various precast members were easily handled on the site (*Fig.* 7).

A bridge with a simply supported span of 125 ft. was constructed using a prestressed concrete beam, shaped so as to be of uniform strength throughout its length, composed of three boxes tied together by the upper slab and by eleven transverse members. The depth of the

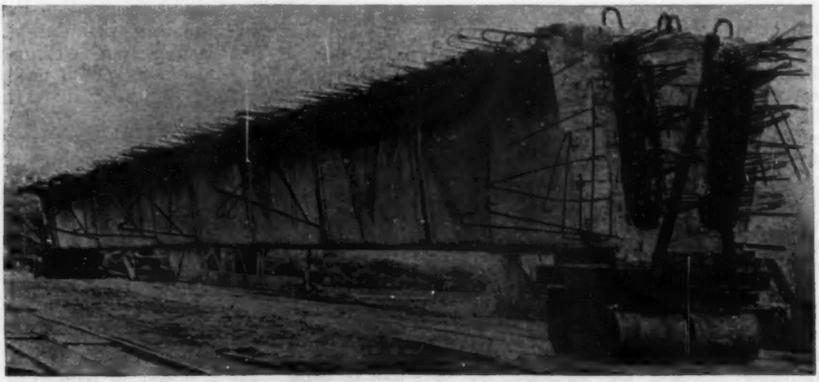


Fig. 7.—Transporting a Beam.

structure is slightly over 6 ft. at mid-span and under 4 ft. at the supports. The high-tensile steel prestressing wires of $\frac{1}{8}$-in. diameter were tensioned to 140,000 lb. per square inch. The abutments are of reinforced concrete. The slenderness ratio of the beams (depth to length) is 0·0475. For each square foot of deck 1·3 cu. ft. of concrete, 5·6 lb. of high-tensile steel, and 5·7 lb. of mild steel were used. The use of simply-supported beams for spans longer than 125 ft. is not economical.

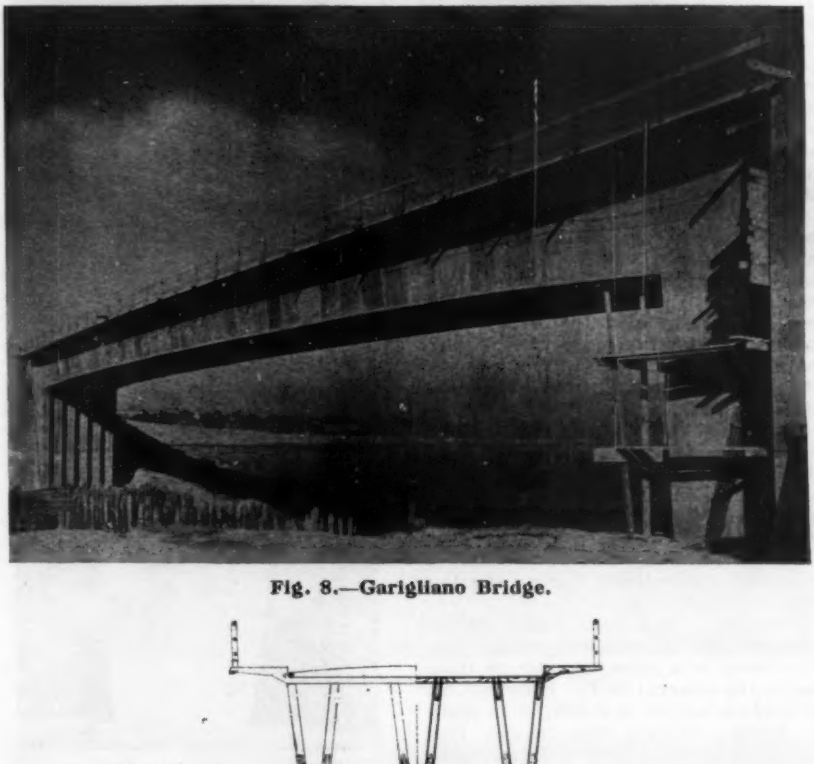


Fig. 8.—Garigliano Bridge.

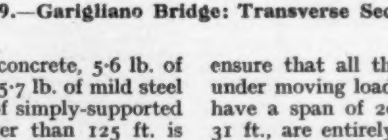


Fig. 9.—Garigliano Bridge: Transverse Section.

Hollow "Box" Girders.

Good foundation conditions allowed the use of a statically-indeterminate structure for the Garigliano bridge, in the Giunture region (*Fig.* 8). The bridge consists of three parallel frames (*Fig.* 9), slightly arched, restrained at the foundations, and with tapering piers (*Fig.* 10). The three frames are tied together by two end-blocks at the piers, by the upper slab, and by the bottom slab near the piers. There are also thirteen transverse slabs which ensure that all the frames act together under moving loads. The frames, which have a span of 200 ft. and a height of 31 ft., are entirely prestressed.

The San Niccola bridge at Benevento is a double cantilever frame with hinges at the base of the piers (*Figs.* 11 and 12). The overall length is 390 ft. with a central span of 260 ft. The supports each consists of eight columns 1 ft. 4 in. thick

and tapering from 13 ft. to 4 ft. 6 in. wide, which are hinged to a cellular base designed to transmit a pressure of 3¾ tons per square foot to the foundation. The deck is supported by four hollow beams, with walls 5 in. thick and varying in depth from 8 ft. 10 in. to 11 ft. 8 in., joined by a top slab 5 in. thick, and over part of the span by a bottom slab. There are also twenty transverse members of varying height and thickness. Concrete with a minimum strength of 6400 lb. per square inch was used. The prestressing steel was high-tensile steel $\frac{3}{16}$ in. diameter (minimum ultimate strength of 256,000 lb. per square inch, and an elastic limit of 214,000 lb. per square inch). The secondary reinforcement consists of round mild-steel bars.

It would be dangerous to prestress a beam of this length after it had been fixed at its supports, since the contraction would cause deformation at the joint between the supports and the beams, and reduce the restraint of the beams. Two temporary rocker bearings were inserted between the supports and the beam so that the beam could contract freely under the prestressing force. With the beam simply supported on the columns, the centering was removed in such a way that, under the permanent loads, the bending moments at mid-span were reduced due to the action of the cantilevers. The rocker bearings were then fixed so that the structure acted as a statically-indeterminate frame under live loads.

By means of tension rods, bending moments can be produced at the ends of a beam in a sense opposite to those due to the external loads. Although the principle is simple, it is difficult to apply to bridges because the tension rods undergo variations in length as a result of the normal deformations of the beams to which they are attached and may become fatigued due to variation in stress. Care must be taken, therefore, in deciding the allowable range of stress. The bridge over the Cerami river in Sicily (*Fig.* 13) is designed so that the loads are transmitted to each support by two connecting members, one in tension and the other in compression. The structure has an overall length of 250 ft.

The bridge over the river Oreto near Palermo (*Figs.* 14 and 15) comprises a continuous prestressed concrete beam resting on oblique elastic members forming two trestle supports, which are hinged upon two bases supported on piles. The bridge has a central span of 182 ft., two side spans each 68 ft. long, and two end cantilevers each 36 ft. long. The width of the roadway is 49 ft. and each pavement is 8 ft. wide.

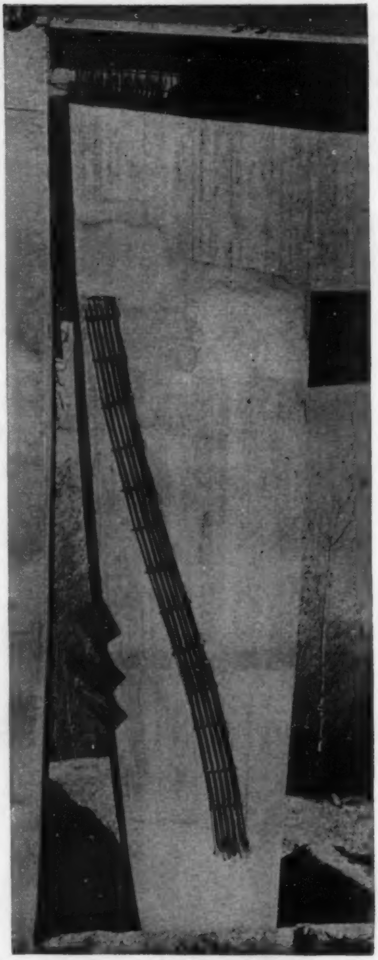


Fig. 10.—Garigliano Bridge: Detail of Support.

Proposed Span of 1300 ft.

The design proposed for a bridge, 5·6 miles long, across the Maracaibo lagoon

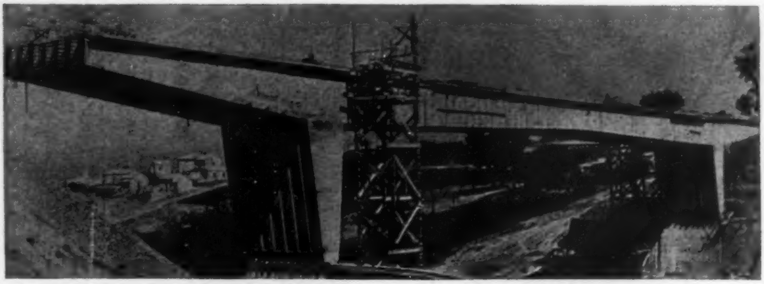


Fig. 11.—San Niccola Bridge, Benevento.

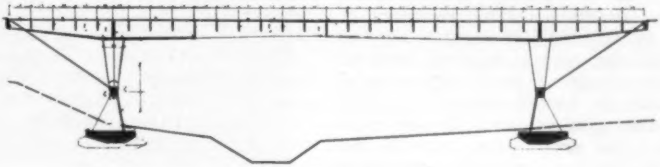


Fig. 12.—San Niccola Bridge, Benevento.

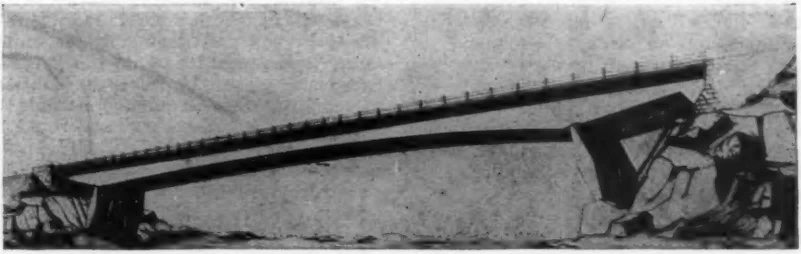


Fig. 13.—Bridge over the Cerami, Sicily.

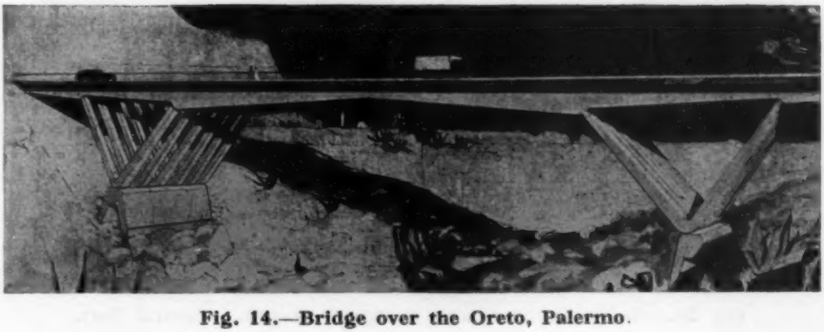


Fig. 14.—Bridge over the Oreto, Palermo.

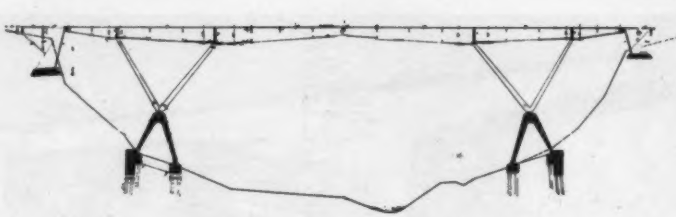


Fig. 15.—Bridge over the Oreto, Palermo.

in Venezuela is shown in *Figs.* 16 and 17. The entire structure would be in prestressed concrete with ten spans of 490 ft. and one span of 1300 ft. All the spans provide a minimum vertical clearance of 150 ft. At the ends of the central section, comprising the central span of 1300 ft. and the two spans of 490 ft. adjoining it, there would be two X-shaped supports. The external arm supports the adjacent span, and the other arm contains the tension ties for both spans and acts as a support. The two extremities of the section, affected by the passage of loads upon the three spans of the section, would be stressed either in compression or tension. The continuous beams would be supported on piers on either side of the central span, and by elastic supports sloping down from two towers 210 ft. high. At the centre a simply-supported beam would be suspended between the ends of the continuous beams and would move independently with variations of temperature.

After the construction of the continuous beam, and while it was supported on centering, two temporary hinges would be formed, as shown in *Fig.* 18, at the central support. Then the tension members of high-tensile steel wires would be tensioned so that the two parts of the beam were raised clear of the centering. In this way the tension members would support the weight of the beam without elongation (except that caused by thermal variation) and there would be no distortion of the central support by the beam because of the temporary hinges. The horizontal component of the oblique force in the cables which pass over the towers would produce a compressive force along the plane of the centre of gravity of the beam which, together with the force produced by the prestressing cables in the same beam, would provide the precompression required. Under these conditions the inclined tension members would be stressed so that the structure would be stable under its own weight. The simply-supported central beam would then be placed in position joining the two balanced systems. After some days, when equilibrium had been reached, the inclined

Fig. 16.—Design for Maracaibo Bridge, Venezuela : Central Part.

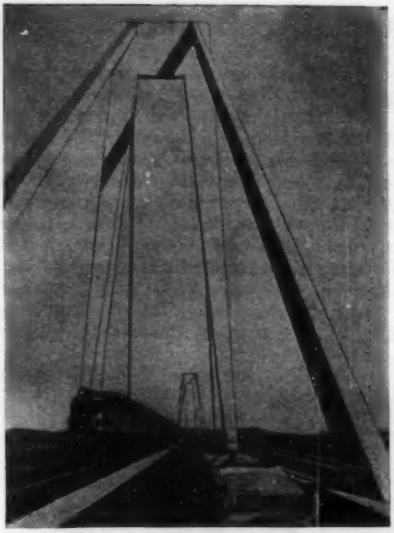


Fig. 17.—Design for Maracaibo Bridge.

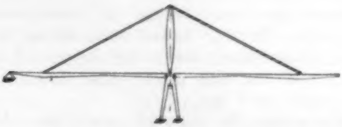


Fig. 18.—Design for Maracaibo Bridge: Central Part.

tension members would be encased in concrete, but the wires could be free to move in their ducts. After this, the cables would be further prestressed in order to compress the concrete sheath, and the cables grouted. Finally the temporary hinges at the central supports would be fixed.

A New Periodical.

In the year 1954 the organisations representing the cement and concrete industries in Denmark, Sweden, Norway, Finland, and Iceland combined to form the Nordisk Betonforbund (North European Concrete Association), and the new association has now produced the "Nordisk Betong" ("Northern Concrete"). The editorial board comprises representatives from Denmark, Finland, Norway, and Sweden. The editor is Mr. Sven G. Bergstrom, of Sweden, and the journal is published at Drottning Kristinas 26, Stockholm. The journal is printed in the Swedish language, with summaries in English, and the annual subscription rate is 20 Swedish kroner.

The first number contains a description by Caspar Trumpy of the Puddefjiord bridge, Norway, with a central arch span of 150 metres in reinforced concrete; details of the standard specifications for concrete and reinforced concrete in Denmark, Finland, Norway, and Sweden, by Viljo Kuuskoski; a paper on torsion in slabs, by Jørgen Nielsen; a paper on the creep of concrete subjected to loads for long periods, by Lars Östlund; and a review of recent research in the countries concerned.

Gold Medal Awarded to M. Eugène Freyssinet.

The gold medal of the Institution of Structural Engineers is this year awarded to Monsieur Eugène Freyssinet, the eminent French engineer, who has taken a leading part in the development of prestressed concrete. This is the first time that the Institution has awarded its gold medal to a non-member. It is hoped that Monsieur Freyssinet will be present to receive the award.

Floors without Projecting Beams.

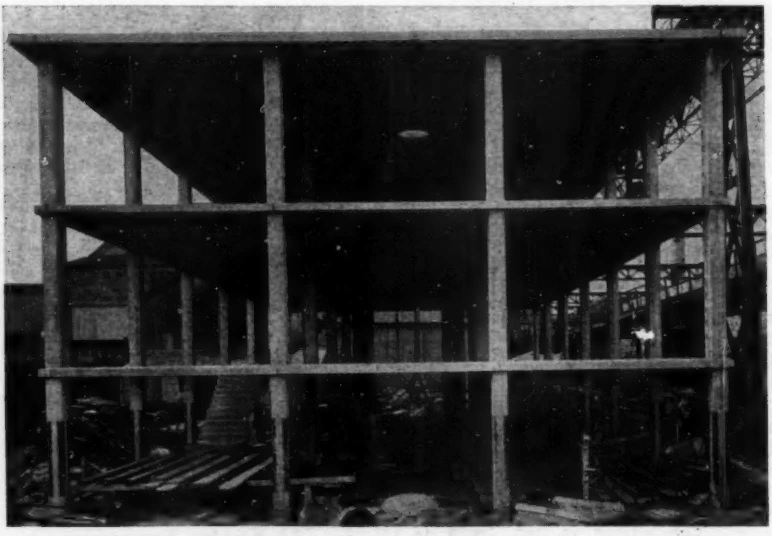
Mr. E. Shepley, B.Sc., M.I.C.E., of the Trussed Concrete Steel Co., Ltd., writes as follows:

Your May number has an Editorial Note which shows how an inexact terminology has arisen in prestressed concrete. In the same issue there is a description of an office building in London, and on the cover the floors are described as "flat-plate".

We must be careful with the terminology of concrete plate construction, because these floors appear to be conventional hollow clay-tile slabs carried by beams of the same total depth, resulting in a flat ceiling. I would restrict the term "plate" to solid construction akin to steel-plate work, where a plate may be flat, folded, dished, curved, domed, corrugated, etc. It is admitted that in the building described the introduction of parallel rows of "pots" forms a ribbed plate, but the loads from the ribs are transferred to the shallow beams and columns in a perfectly standard fashion, and the floor can hardly be described as a flat plate. A plate could, of course, be ribbed to improve its buckling resistance, but in this case the ribs would be made relatively small and would not be independently supported—for the very purpose of avoiding beam action.

It is considered inherent in plate design that the loads find their way directly to the main supports by natural paths normal to the contours of deflection, whereby shear, torsion, bending and direct actions are compounded into a minimum entropy state which permits the engineer to design with the utmost economy of both steel and concrete.

Several examples of folded-plate construction have been illustrated in your journal and, of course, all shell construction would come under my definition as curved-plate work. True flat-plate designs for multiple story flats and office buildings have been constructed by the Trussed Concrete Steel Co., Ltd., by their Truscon Plate System, and I believe one or two buildings have been constructed with plates of variable thickness which have been referred to as "taper plates".



The illustration is of an industrial office building, designed for an imposed load of 50 lb. per square foot, in course of construction by the Truscon Plate System. The columns are 10 in. by 10 in. in cross section and the floors $5\frac{1}{2}$ in. thick.

Prestressed Concrete Bridge for Heavy Loads.

By E. A. L. KEITH, B.Sc.(Eng.), A.C.G.I., A.M.I.C.E.

THE preliminary civil engineering work for the new power station now being built at High Marnham, Notts, included the construction of a bridge (*Fig.* 1) to carry a road over new railway sidings to the power station. As this road is the most suitable approach to the power station for transporting the stators and transformers, allowance had to be made for very heavy loads.

A transformer and its transporter, which has 3-axle bogies, together weigh about 240 tons, thus exceeding the 45-unit highway-bridge loading. For a stator and its transporter, which together weigh 210 tons, the load is only slightly less as the transporter has 2-axle bogies. Since these abnormally heavy loads will probably occur infrequently, the bridge was designed for the 45-unit Ministry of Transport highway-bridge loading so that the pre-compression in the bottom flanges of the beams would be zero under this loading, and an allowance was made for temporary overloading by abnormal loads. This design ensures compression in the bottom flanges of the beams under normal Ministry of Transport loading, which is considered desirable for bridges over railways. The bridge will also be adequate to meet future requirements if, as seems probable, this road is given a higher classification.

Although the existing road was re-aligned and re-levelled, the revised clearance for over-bridges of 15 ft. 6 in. required by British Railways seriously limited the depth of construction of the deck. Several schemes were considered, including a composite girder and reinforced concrete slab, but the most economical and speediest form of construction was found to be prestressed precast beams with post-tensioned steel with concrete infilling and slab as shown in *Fig.* 2.

Fig. 1.

The deck is composed of twenty-seven beams, nearly I-shape (*Fig.* 2), placed side by side with the ¼-in. nibs touching. The spaces between the bottom flanges were filled with cement mortar, the nibs acting as shuttering, and then concrete was placed to fill the spaces between beams and form a slab 3 in. thick above the top flanges. Two sheets of saturated bituminous felt were placed on this layer of concrete, and the top layer of concrete was formed to the correct camber, with longitudinal falls towards the road gulleys at the ends of the approaches. The beams (*Fig.* 7) were made in a factory, and were prestressed by the Gifford-Udall system ; Lee-McCall high-tensile bars were used for the transverse prestressing.

The span is 34 ft. 4¾ in. and the width between the parapet walls 45 ft., the roadway occupying the central 22 ft. The bridge has an angle of skew of about 20 deg., so that, contrary to the usual practice, transverse stressing could only be arranged satisfactorily parallel to the abutments. Correct alignment of the beams was important, and it was therefore decided to tension only two wires of each cable at a time to avoid any lateral deformation. In addition, the ¼-in. nibs at the sides of the beams were omitted and the beams made ½ in. narrower to provide

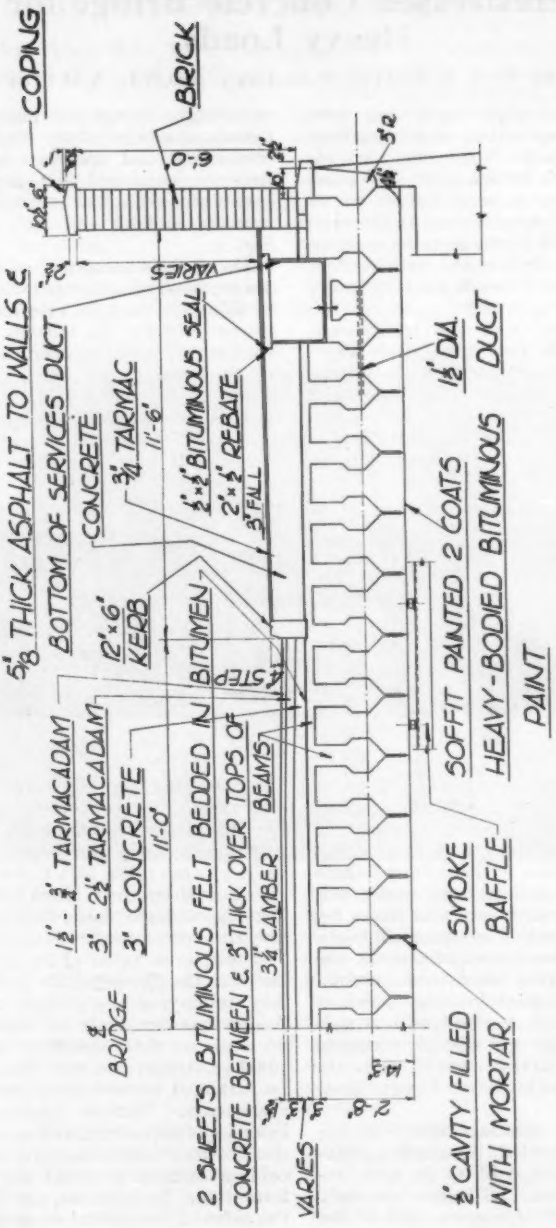


Fig. 2.—Cross Section.

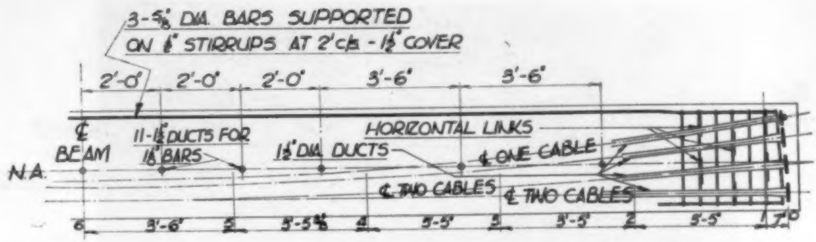


Fig. 3.—Sectional Elevation at Centre-line of Beam.

greater tolerance if lateral deformation occurred. The beams were true after prestressing, and there was no difficulty in seating and aligning the transverse ducts.

Ducts for services were formed on both sides of the bridge by using shallower beams adjacent to the parapet beams. These ducts were lined with asphalt ⅜ in. thick, and covered with precast slabs over which the tarmacadam of the pavement will be carried after the services have been laid in the ducts. The cross sections of the beams are similar. Of the 27 beams used, 23 are identical; the beams under the ducts have a narrower top flange, and the parapet beams are wider and have a rounded arris at the outer bottom edge to facilitate the escape of smoke (*Fig.* 8). Thus only three types of beams were used and the same moulds were used for all of them. The use of similar beams under the roadway and the pavements makes it possible to widen the roadway, if required, without difficulty.

The bearing-plates are of special interest. The phosphor-bronze plates are recessed in the main steel body of the top plates to prevent them working loose. A transverse boss in the top plate and a corresponding slot in the bearing surface of the bottom plate effectively anchor each beam laterally.

Details of a fixed bearing are shown in *Fig.* 5. The sliding bearings are very similar in construction but provide tolerance to permit longitudinal movement and deflection. Further details of the seatings at the abutments are shown in *Fig.* 6. An angle bolted to the soffits of alternate beams near the ends encloses the bearings completely and thus prevents fouling by birds. Smoke-baffles are suspended from the underside of the deck above the centre-line of each track. The whole of the underside of the bridge, including the

Fig. 4—Beams in Position.

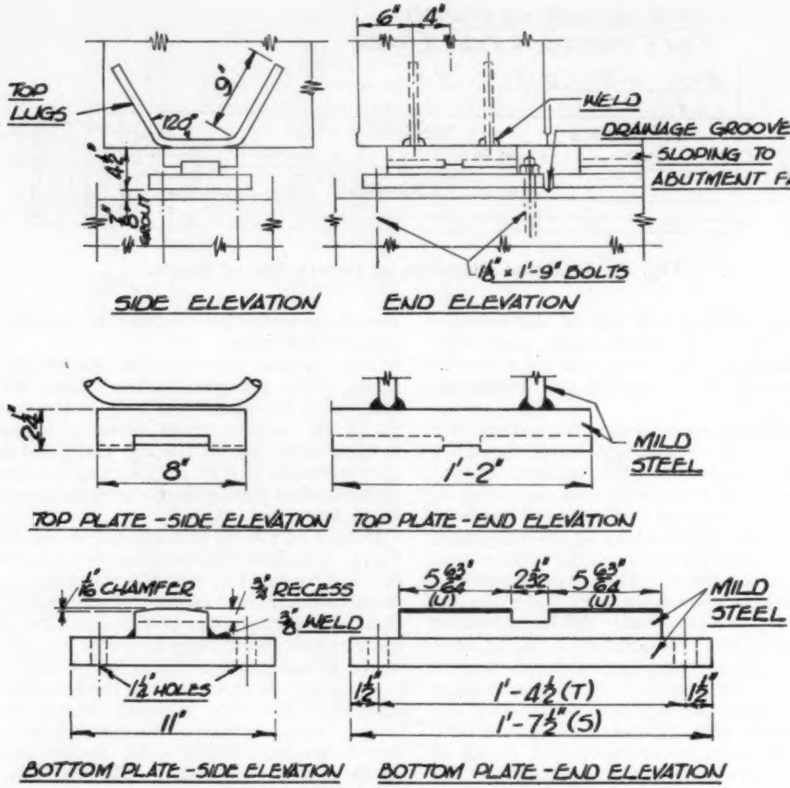


Fig. 5.—Details of Fixed Bearings.

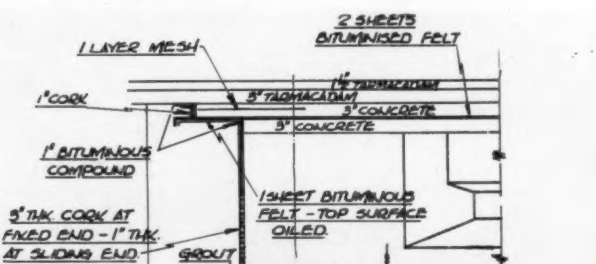


Fig. 6.—Details of Seating at Abutment.

Fig. 7.—Beams Ready for Erection.

Fig. 8.—A Parapet Beam.

baffle-plates, the protection plates, and all exposed surfaces of bearings, nuts, bolts, etc., were painted with two coats of heavy-bodied bituminous paint.

A cornice was cast in place to cover the anchor-plates and anchor-nuts of the transverse stressing bars; a precast cornice above this feature was continued at the top of the wing walls. In the abutment and wing walls, trapezoidal horizontal grooves were arranged at regular vertical intervals to hide the construction joints between successive lifts, and the surface was bush-hammered to within an inch above and below these grooves. At a late stage in the work a re-alignment of the sidings resulted in the tracks being curved under the bridge, and it was necessary to corbel the eastern half of the north abutment in order to provide the specified clearance. This corbel is visible only from the cutting for the station sidings and has not detracted from the appearance of the bridge.

The cost of the works, including abutments, wing walls, road surfacing, parapets, etc., was about £4 12*s*. 6*d*. per square foot. The clients were the Central Electricity Authority, and the bridge and the roadworks associated with it have been taken over by the Nottinghamshire County Council. The contractors were Messrs. M. J. Gleeson (Contractors), Ltd., and the beams were made by the Kingsbury Concrete Co., Ltd., at their works about ten miles distant from the site. The consulting engineers were Messrs. Freeman, Fox & Partners.

Design of Silos.

THE design of deep beams for walls of silos, using methods devised by Dischinger, is explained in a recently-published German book.* From the diagrams and tables, the distribution of stress σ_X, the total tension T, the lever arm d, and the distance d_0 from T to the lower edge of the beam (*Fig.* 1) can be easily obtained for a continuous girder, with an infinite number of spans, for the usual types of loading. The values of T, d, and d_0 for single-span beams can be calculated, using the special case in which the width of the support is equal to half the span.

Navier's straight-line law appears to apply to deep girders, provided that the depth of the beam does not exceed two-fifths of the span for single-span girders, or four-fifths of the span for continuous girders. Above these values the distribution of stress deviates considerably from the straight-line law.

In silo walls, beam action is shown to occur mainly near the lower edge. In the case of a continuous girder the effective depth can be assumed to be about equal to the span.

The total tension was found by integrating the tensile stresses, and it was established that when $\frac{H}{L}$ is less than two-fifths the lever arm of the internal forces is proportional to the depth of the beam, but when $\frac{H}{L}$ is greater than or equal to 0·45 the lever arm is a function of the span.

* "Hilfstafeln zur Berechnung Wandartiger Stahlbetonträger." By Otto F. Theimer. (Berlin: Wilhelm Ernst & Sohn. 1956. Price 7.20 D.M.)

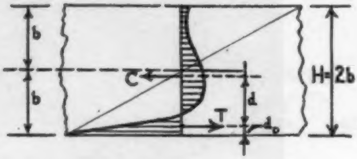


Fig. 1.

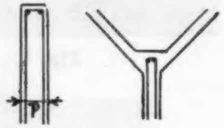


Fig. 2.

It is considered wrong to stiffen the walls of silos by providing additional edge beams; the reinforcement in such beams is stated to be ineffective because of the stiffness of the wall; the arrangements shown in *Fig.* 2 are recommended.

An important conclusion is that the top reinforcement over the supports of deep beams should not be at the upper edge of the beam, but in a position determined from the diagrams, as shown in *Fig.* 3; in this illustration a represents nominal reinforcement, at b are indicated suspended bars if the load is applied below (as in hoppers), C indicates small bars to resist shrinkage and secondary stresses, and D is the main reinforcement.

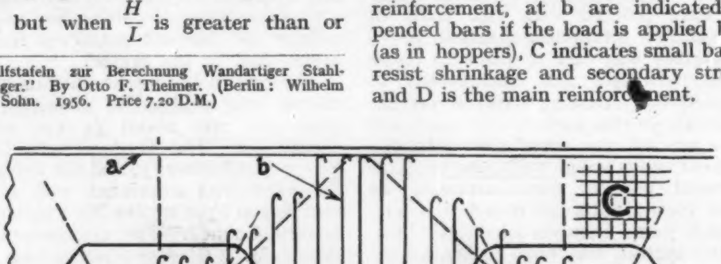


Fig. 3.

The Terminology of Prestressed Concrete.

MR. JAMES N. LOWE, M.A., A.M.I.C.E., of the Pre-Stressed Concrete Co., Ltd., London, writes as follows.

SIR,—In your Editorial Notes for May 1957 you do a service in drawing attention to the confusion of thought in the terminology used in prestressed concrete and, I suspect, use some deliberate exaggeration to impel your readers into replying. I feel sure you will succeed in your aim.

First of all, may we define the process we wish to name as " the application of a system of forces to a structure or structural member before it is called upon to withstand working loads ". The reasons for applying the process are numerous, and vary from fundamental structural reasons to useful tricks. In applying the process to a medium, we choose the system of forces to restrict the stresses at any time to those which the medium will accept. In applying the process to concrete, therefore, we limit the total compressive stress to that acceptable and permit very little or probably no total tension.

Therefore, before applying working loads we apply forces, and these forces cause stresses which are both compressive and tensile. Hence a logical name for the process is " pre " (that is before applying working loads) plus " stressed " (that is the result, both tensile and compressive, of applying forces). Hence the word prestressed. Indeed the incorporation of the term " stressed " is a healthy reminder that the application of this process causes all forms of stress, including bending, shearing, and torsional, positive and negative, and it invites the engineer to ensure that at all stages these kinds of stress are within the capacity of the particular medium. I submit therefore that " prestressed " is short and pithy and is understood, and is therefore acceptable.

The next stage is to find names for the methods of applying these forces, bearing in mind that over the years we shall see great changes. How often nowadays do we refer to the Hennebique System ?—we talk about reinforced concrete. We may well see a similar change in prestressed concrete, but the principle of prestressing will be applied in many ways and for many new purposes, so we must not presume too much in giving names now or we may cause confusion later.

The particular terms we want at the moment are for use where the system of forces applied is the result of the position, the shape, and the load of high-tensile steel wires or bars. We want to differentiate between the two basic methods of doing this. The two methods vary in the time when concrete is cast, in the way in which the steel passes its forces to the concrete, and in the consequent techniques of design.

In one method the tension of the steel is maintained by anchoring its ends with devices cast in the concrete ; its position is maintained likewise ; the shape is maintained by ducts cast in the concrete. In this case the steel is tensioned after the concrete has matured.

In the other method the tension in the steel is maintained by bond with the concrete ; its position is maintained likewise —so far it does not seem to be practical to impose a shape. The steel is tensioned before the concrete is cast, the tension being temporarily resisted by abutments, and this force is transferred from the abutments to the concrete, to which it is bonded, after the concrete has matured.

Thus the two methods are seen to have many complex differences : the time of casting the concrete, the time of tensioning the steel, the shaping of the steel, the method of transferring the tension in the steel to the concrete, and more are involved. The words to describe these two methods should be simple and short, should indicate complexity, and should be chosen for use by the expert.

The immediate differentiation between the methods can be achieved by using " pre " and " post " to indicate whether the steel is tensioned before or after the concrete has been cast and matured. The word " tensioned " is added, giving " pre-tensioned " and " post-tensioned ". These portmanteau adjectives should, of course, be applied only to the steel. For example, we may speak of prestressed concrete and say that the prestressing is achieved by a method using post-tensioned steel or by a method using pretensioned steel. These are the three portmanteau words, the one indicating

simplicity of thought and the other two indicating complexity of method. It is necessary to have a word rather than a long phrase, not only for simplifying writing but also for simplifying specifying, billing, and both private and public speaking.

I believe too that the term " at transfer " is both simple and accurate. It refers to the transfer of anchorage from abutments to concrete bond, and calls upon the engineer to exercise all that care which he would exercise when transferring a load from one support to another.

I think there are two more important pieces of confusion that you refer to. It is dangerous, illogical, and useless to refer to tensioned steel as reinforcement. The prestressing steel is an external system of forces which is active. Reinforcement is a passive potential supply of tension in a medium unsuited to resist such tension. Your use of tensioned steel and untensioned steel for prestressed concrete and reinforcement for reinforced concrete are sound.

You are also right to advocate referring to abutments for pre-tensioned steel, though I prefer " anchorages " to " fixings " for post-tensioned steel because fixings occur in other parts of engineering and building.

I find it very difficult to provide for myself a universal terminology for the parts of the anchorages and the equipment for the various systems, but strongly believe that the years will see a simplification—perhaps an elimination—of method, as we saw in reinforced concrete, and with some cowardice I leave further suggestions until that time.

[We are pleased to see that our correspondent agrees with all our suggestions except the use of the word precompressed instead of prestressed. As he suggests, precompressed is probably more accurate, because the purpose of prestressing is to " permit very little or probably no total tension", but the word prestressed is now too well entrenched for a change to be made.

We do not agree that technical terms " should be chosen for use by the expert " only. They should also clearly express their meaning if we are to meet the common complaint that scientists and technologists use so much jargon that they cannot be understood by others, and which may mean the opposite of their everyday meanings. We see no reason why such terms should indicate complexity, or why it should be necessary for a designer to use a term which will indicate to him that he is dealing with a complex problem—if he does not know that he should not be allowed to design in prestressed concrete. Nor do we believe that the mere use of the meaningless " at transfer " reminds the engineer " to exercise all that care which he would exercise when transferring a load from one support to another ". If we say " when the prestress is applied " we describe exactly what we mean, and equally well remind the engineer that a force is transferred from the steel to the concrete.—EDITOR.]

Reinforced Concrete for Government Buildings.

REPLYING to a question in the House of Commons recently, the Minister of Works stated that it was estimated that the amount of steel used in building last year was 1¼ million tons. Reinforced concrete was now generally adopted for Government buildings unless there was a good reason to the contrary. During the year ended June, 1956, some 160 framed buildings of all types were designed by the Ministry, and of these 110 were in reinforced concrete. In the case of the second half of the new Government buildings in Whitehall Gardens, London, reinforced concrete was being used instead of structural steel and as a result there was a saving of £130,000.

Lectures on Road Materials.

LECTURE courses dealing with the properties of road materials and the application of the results of research will be held at the Road Research Laboratory, Harmondsworth, during the autumn and winter of 1957–58. Full details can be obtained from the Director, Road Research Laboratory, Harmondsworth, West Drayton, Middlesex.

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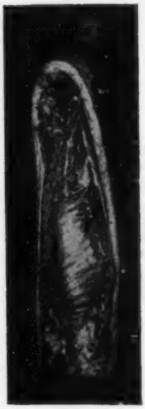

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Aggregate from Pulverised-fuel Ash.

THE first plant in this country for the production of concrete aggregate from pulverised-fuel ash is now in operation, by the Cementation Co., Ltd., at the Battersea (London) power station of the Central Electricity Authority, under licence from Sinterlite, Ltd., and Dr. F. P. Somogyi, the inventor of the process. The cost of the main plant (*Fig.* 1) was £260,000.

The amount of this ash that has to be disposed of from power stations in Great Britain is nearly 4,000,000 tons a year, and the cost of its disposal is about £1,000,000 a year.

The ash from the electrostatic precipitators is first delivered into bunkers from which it is delivered into a shallow circular pelletiser, revolving at an angle, where it is sprayed with water. In the revolving pelletiser, gravity and centrifugal force cause the damp ash to form into small balls, the size of which is regulated by the speed of revolution and the angle of tilt of the pelletiser. The pellets spill over the rim of the pelletiser into a conveyor which takes them to the kiln.

The kiln measures about 40 ft. high by 10 ft. diameter and is of welded steel plate lined with refractory bricks. The temperature, which attains a maximum of about 1400 deg. C., depends upon the draught in the kiln, and is regulated by instruments linked with thermocouples inside the kiln. Each kiln has an annual capacity of about 50,000 tons of aggregate.

As there is enough carbon in the ash for combustion, no fuel is needed other than that required to kindle the fire at the start. In the burning process the carbon and any sulphur and other volatile compounds are driven off, rendering the final product inert. The process is continuous, the pellets slowly descending from the top of the kiln until they reach a revolving grate at the base through which the sintered pellets emerge to be screened, graded into sizes, and stored in hoppers.

Fig. 1.—Plant for producing Aggregate from Pulverised-fuel Ash.

The aggregate, which is known as Terlite, is made in three sizes, namely from $\frac{3}{4}$ in. to $\frac{3}{8}$ in. weighing 48 lb. per cubic foot, $\frac{3}{8}$ in. to $\frac{3}{16}$ in. weighing 50 lb. per cubic foot, and $\frac{3}{16}$ in. to dust weighing 52 lb. per cubic foot. Concrete made in

the proportions of 1 part cement to 8 parts aggregate without fine material weighs 68 lb. per cubic foot and has a compressive strength of 600 lb. per cubic foot. A mixture of 1 part cement to 2 parts fine aggregate and 3 parts of the larger aggregate weighs 92 lb. per cubic foot and has a compressive strength of 3000 lb. per square inch. The shrinkage on drying and moisture movement are each about 0·025 per cent., and this type of concrete has good properties of insulation and resistance to fire. It is nailable, and provides a key for plaster.

The most suitable water-cement ratio (including any water in the aggregate) is about 1·1 to 1·2. For example, similar mixtures had a compressive strength of 1750 lb. per square inch with a water-cement ratio of 0·9, 2400 lb. with a water-cement ratio of 1·0, 2800 lb. with a water-cement ratio of 1·15, and 2400 lb. with a water-cement ratio of 1·3.

Tests have shown that concrete with 400 lb. of cement per cubic yard has a compressive strength of about 1200 lb. per square inch at 28 days; with 600 lb. of cement the strength is about 2800 lb.; with 800 lb. of cement, about 3750 lb.; with 1000 lb. of cement, about 4250 lb.

[Further notes on this type of aggregate were given in this journal for December 1956.]

Single-story Industrial Buildings.

A BROCHURE entitled "Single-story Industrial Buildings", issued by the Cement and Concrete Association, describes forty-five such structures, including structures with shell and north-light "shell" roofs and pitched and Mansard roofs of prestressed and reinforced concrete, both precast and cast in place.

The most common type of building comprises gable frames of precast reinforced concrete, but a warehouse at Hayes, Middlesex, has reinforced concrete frames spanning 89 ft. that were cast in place at 19 ft. 2 in. centres. The rafters are 4 ft. 3 in. deep at the column, tapering to a hinge at the crown.

A factory at Lowestoft (*Fig.* 1) has a north-light shell roof with a span of 86 ft. The valley-beams are supported on columns at 26 ft. 6 in. centres, and the valley-beams and upper edges of the shells are prestressed (the architects are Messrs. Buckingham & Berry, the engineers Twisteel Reinforcement, Ltd., and the contractors Messrs. R. G. Carter, Ltd.). On the other hand, a factory in Yorkshire is covered by a shell roof spanning 88 ft. with reinforced concrete valley-beams at 42 ft. centres. The brochure can be had free from the Cement and Concrete Association at 52 Grosvenor Gardens, London, S.W.1.

Fig. 1.

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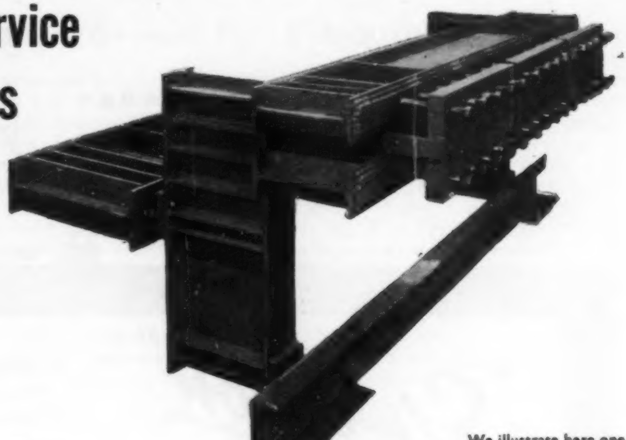

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Consolidation by Vibration.

A JOINT committee was appointed by the Institutions of Civil and Structural Engineers in the year 1935 to consider the compaction of concrete by vibration. Interim reports were published in the Journal of the Institution of Civil Engineers for March 1937 and April 1938, and a report * recently published is more of the nature of a manual on vibrated concrete for the use of engineers and contractors.

After summarising the objects and uses of vibration, the committee considers how the efficiency of the process is affected by the characteristics of the materials, the proportions, and the shape and grading of the aggregates. It is recommended that coarse aggregates be ordered in different sizes so that they may be stock-piled and used to maintain constancy of grading, which is particularly important in leaner mixtures. Stock-piles should be arranged to avoid segregation of the coarse aggregate and to ensure uniform water content in both coarse and fine material. Batching by weight, compensation for variations in the water content of aggregates, apparatus for measuring workability, and the use of the compacting-factor apparatus are recommended.

Nearly half of the report deals with the selection of aggregates and mixtures made with continuously-graded aggregates and gap-graded aggregates. For continuously-graded aggregates the recommended method is that given in "Design of Concrete Mixes" (Road Note No. 4 of the Road Research Laboratory), modified slightly to suit vibrated concrete. The principal modification is the omission of the finest grading zone, leaving only three grading curves with two zones between them. These curves are otherwise the same as those in Road Note No. 4, but in Fig. 6 of the report, which gives gradings for 1½-in. aggregates, one gap-grading is added. The other tables are similar to those in Road Note No. 4 except that grading No. 4 is omitted from the relevant tables.

The steps in obtaining a suitable mixture with continuously-graded aggregate are as follows. (1) Knowing the minimum strength required and the efficiency of the works control to be used, determine, with the help of Table 1 and Fig. 4, the average strength required and the corresponding water-cement ratio. (2) Use Table 2 to determine the degree of workability ("very low", "low", "medium" or "high"). (3) Use Table 3 or 4 to determine the aggregate-cement ratio and the most suitable grading. (4) From the sieve analyses of the proposed aggregates calculate proportions of the fine and coarse material that will produce a combined aggregate with a grading as close as possible to the selected curve. (5) Since the water-cement ratio and the aggregate-cement ratio have been obtained, all the proportions can now be decided. (6) A trial mixture on the site may show a need for small modifications in the calculated proportions, for example a small increase in the amount of mortar. Fully-worked examples are given to illustrate this procedure.

In the case of gap-gradings a theoretical method is recommended based on filling the voids in the coarse aggregate with mortar made with fine aggregates small enough to enter the voids in the compacted coarse aggregate; the result of the calculation must be checked by trial mixtures. Other recent work, however, suggests that this rather elaborate calculation might be avoided by determining the most suitable continuous grading and deriving the gap-grading from this; trial mixtures, which must be made in any case, indicate any modifications necessary.

Mixtures richer than 1 : 4 by weight are dealt with separately. The increase of strength with cement content for such concretes of constant workability is very much less than in the case of leaner concretes. Diagrams show the variations of crushing strength with the compaction factor for proportions of 1 : 2½ and 1 : 3½ with ¾-in. aggregate, and the variations of strength with water-cement ratio for 1 : 3 mixtures with ⅜-in. and ¾-in. aggregates. The necessity for making trial mixtures with the selected proportions is emphasised.

There are paragraphs on shuttering and on mixers, the latter stating a need for a new type of mixer specially designed for

* "The Vibration of Concrete." Published by the Institutions of Civil and Structural Engineers. 57 pages. Price to members 3s. 6d.; to non-members, 8s.

stiff lean mixtures. Most of the remainder of the report is a discussion of immersion, clamp-on, table, and other vibrators, their methods of drive, characteristics, suitabilities and general use, and methods of placing concrete by vibration. Special attention is recommended in the use of surface vibrators on roads and runways so as to ensure a surface with good riding properties. Suggestions are made for further research. A very useful section gives recommendations for clauses of a specification for vibrated concrete. These would be additional to the normal clauses, and are intended to ensure the adequate control of the materials and their proportions, the adequacy of samples, the tightness and freedom from distortion of shuttering, the choice of vibrators, the control of vibrators, and the control of the strength of the concrete.

The report is a compact handbook on vibrated concrete, concise enough for quick reference and yet full enough for ordinary use, and will be useful to site engineers and inspectors as well as to designers of concrete works.—H. N. W.

Change of Title.

THE title of Twisteel Reinforcement, Ltd., has been changed to G.K.N. Reinforcements, Ltd., in order to indicate its connection with the Guest, Keen & Nettlefolds group of companies and because the former title does not describe the work and products of the company, which now supplies all types of reinforcement. When the company was formed in 1924 it undertook the supply on a limited scale of square twisted bars and woven fabric. Its products now include Wireweld fabric, Tentor bars and fabric, prestressing wire, and round mild steel bars, and it also undertakes the design of concrete structures.

New Immersion Vibrators.

NEW immersion vibrators, known as "200 Cycle" machines, are now available from Messrs. E. P. Allam & Co., Ltd. The motor is within the vibrator, so dispensing with flexible leads. The electricity may be generated 100 yd. from the position where the vibrators are used. All the motors operate at low voltage, 200 cycles, three phase, and are made to withstand high temperatures. The standard sizes are 2 in. diameter by 17½ in. long, 2½ in. diameter by 18¾ in. long, 3½ in. diameter by 18 in. long, and 1¼ in. diameter by 35 in. long.

Anglian Building Products, Ltd.

Messrs. Boulton & Paul, Ltd., and Anglian Building Products, Ltd., announce that an agreement has been concluded by which Messrs. Boulton & Paul, Ltd., will take a financial interest in Anglian Building Products, Ltd., and its associated companies, Norwich Ready Mixed Concrete, Ltd., and Atlas Sand & Gravel Co., Ltd.

A Manual of Prestressed Concrete.

A BROCHURE on the use of post-tensioned steel bars in prestressed concrete has been issued by McCalls Macalloy, Ltd., of Templeborough, Sheffield. The contents deal with design, detailing, tensioning the steel, and the application of this type of prestressed concrete to beams, frames, bridges, piles, caissons, tanks, and the strengthening of beams. A number of tables give some useful data, and in an appendix are described methods of making high-strength concrete. Copies of the brochure are obtainable free on application.

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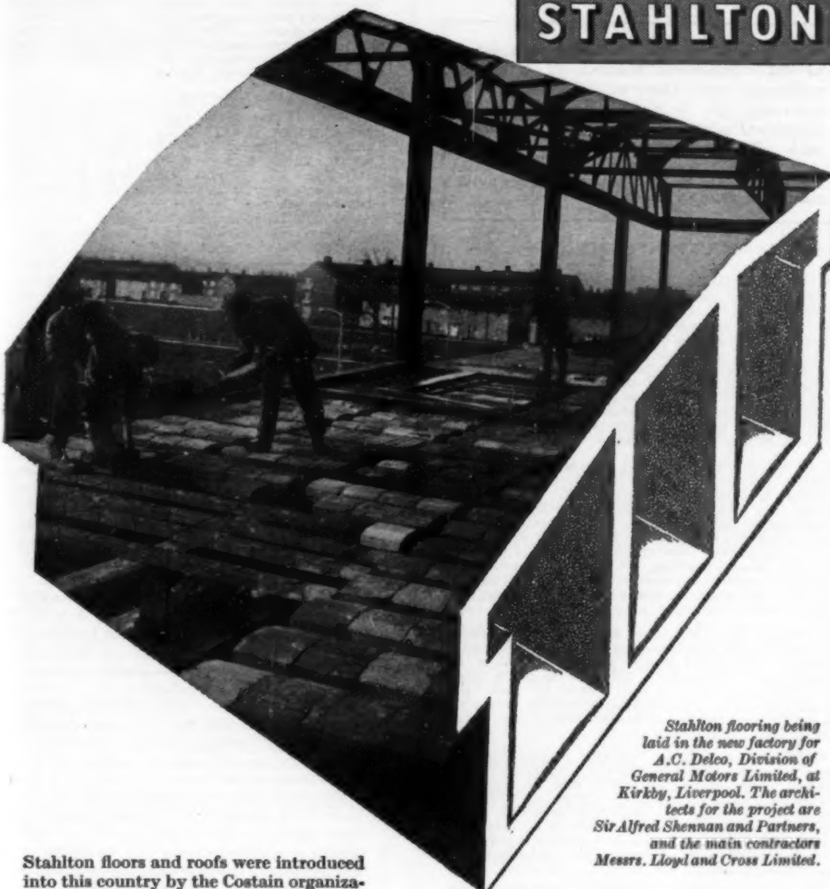
STAHLTON



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(*Continued on page lxxix.*)

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(*Continued from page lxxvii.*)

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(*Continued overleaf.*)

(*Continued from previous page.*)

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Applications are invited for the appointment of CIVIL ENGINEERS at the RESEARCH LABORATORIES, LEATHERHEAD, SURREY, to carry out investigations into the properties of pulverised-fuel ash concrete, the applications of such concrete to civil engineering projects, and the further use of pulverised-fuel ash. Candidates should have a degree or H.N.C. in Civil Engineering, preferably with some practical experience. Salary £550–£995 per annum according to qualifications and experience. Applications stating age, experience, present position, and salary should be forwarded to D. Moffat, Director of Establishments, Winsley Street, London, W.1. Quote reference C.C.E./289.

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A progressive, rapidly-expanding Company, specialising in the manufacture and erection of precast prestressed concrete structures, invites applications from young Engineers for THREE appointments of Assistant. Applicants should be professionally qualified at least in part by University Degree or H.N.C. and/or graduateship of an appropriate Chartered Institution. Exceptions may be considered in certain circumstances of outstanding experience. In addition to a thorough grounding of design and detailing of concrete structures generally, the following qualifications together or singly will be considered an advantage: (a) Practical experience on the site (b) Quality control of concrete through the medium of a Laboratory (c) Public relations as technical advisor.

Applications in *writing only*, stating age, education, marital status, qualifications, experience, with names of references, should be made in the first place to Sumner, Harker & Co. Ltd., 196 Deansgate, Manchester, who will arrange for an interview in August with a Director from Rhodesia.

Salary offered will be not less than £100 per month with annual increments on the basis of a three years' contract.

Free air passage will be given to the Engineer, and assistance to his wife and family. Duties to commence early in October, 1957.

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JOHN MOWLEM & CO., LTD., HAVE A VACANCY IN THEIR ORGANISATION IN PERSIA FOR A STRUCTURAL ENGINEER FOR BRIDGE DESIGN. Applicants must be qualified and thoroughly experienced, particularly in the design of concrete structures and bridge work. Salary inclusive of allowances equivalent to approximately £3500 per annum.

THE TERM OF EMPLOYMENT FOR THE ABOVE VACANCY WOULD BE FOR A PERIOD OF TWO YEARS, RENEWABLE BY MUTUAL CONSENT. APPLICANTS SHOULD APPLY IN WRITING, GIVING BRIEF DETAILS OF THEIR EXPERIENCE, TO JOHN MOWLEM & CO., LTD., 123 VICTORIA STREET, LONDON, S.W.1.

SITUATIONS VACANT. Square Grip Reinforcement Co., Keynsham, Bristol, have vacancies at Bristol and Birmingham for reinforced concrete designers to act as section leaders. Work covers a wide range of structures, and applicants should have had good previous experience. Good salary, profit-sharing scheme, pension, and five-days' week. Modern office in pleasant surroundings. Vacancies also for experienced detailers. Write in confidence, giving full details.

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Vacancies exist with the Engineering Branch of the Industrial Group of the United Kingdom Atomic Energy Authority at its Headquarters at Risley, Warrington, Lancs.

DUTIES:

(a) *Ref. 1898.* To lead a small team of staff engaged on design of domestic, process and cooling water systems, drainage, roads, railways and building services associated with the Authority's construction programme, or to undertake individual design of a good standard

(b) *Ref. 1958.* The preparation of calculations and detailed drawings for all types of structures in steel and/or reinforced concrete under the supervision of a structural engineer.

QUALIFICATIONS & EXPERIENCE:

Applicants must have:

For Post (a) an honours degree in Civil Engineering followed by about 2 years' formal training or practical experience, and be taking active steps towards Corporate Membership of the Institution of Civil Engineers.

For Post (b) a Higher National Certificate or equivalent qualification in Structural or Civil Engineering, and have had experience in the type of work involved.

Candidates should be at least 25 years of age, although younger candidates may be considered.

SALARY: will be assessed within the scale £795 to £1,210 per annum.

A contributory Pension Scheme is in operation.

Authority houses for renting by successful married candidates may be available in due course, or alternatively, substantial assistance may be given towards legal expenses incurred in private purchase.

Send a postcard for an application form, quoting the appropriate reference, to Recruitment Officer, United Kingdom Atomic Energy Authority, Industrial Group Headquarters, Risley, Warrington, Lancs.

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